SR 307 MP 1.45 Northeast Dogfish Creek to Dogfish Creek (WDFW ID 991572): Preliminary Hydraulic Design Report

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ROLES AND RESPONSIBILITIES FOR THIS PHD

The roles and responsibilities of the key individuals in developing this Preliminary Hydraulic Design (PHD) are defined as follows for the Olympic Region GEC:

PHD Lead PE
Responsibility: Water Resources Professional Engineer in responsible charge of this Hydraulic Design Report, including all information, calculations, assumptions, modeling, professional judgment, and commitments contained in the main report and appendices.

Authoring Firm PHD QC Reviewer(s)
Responsibility: Qualified independent individual(s) responsible for the detailed checking and reviewing of hydraulic and stream design documents prepared by the authoring firm, including all information, calculations, assumptions, modeling, professional judgment, and commitments contained in the main report and appendices. Before submittal to the GEC, the authoring Firm Quality Control (QC) Review shall be performed in accordance with the QC methods identified in the quality assurance document Technical Verification Form (TVF). The QC methods are defined in the Olympic Region GEC Quality Management Plan (QMP) Section 5.3 and the QMP Supplement developed specifically for Y-12554 Task AC.

Olympic Region GEC Fish Passage/Stream Design Advisor
Responsibility: Water Resources Professional Engineer providing mentorship, process oversight, quality check issue resolution, and recommendations in the approach to hydraulic analysis and design performed by the **PHD Lead PE**. Before submittal of draft deliverables from the GEC to either the PHD Lead or WSDOT Headquarters, the Olympic Region GEC Fish Passage/Stream Design Advisor will review and refine GEC comments and confirm GEC comment resolution by the **PHD Lead PE**.

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1 Introduction

To comply with United States et al. vs. Washington, et al. No. C70-9213 Subproceeding No. 01-1 dated March 29, 2013 (a federal permanent injunction requiring the State of Washington to correct fish barriers in Water Resource Inventory Areas [WRIAs] 1 through 23), the Washington State Department of Transportation (WSDOT) is proposing a project to provide fish passage at the State Route (SR) 307 crossing of Northeast Dogfish Creek at milepost (MP) 1.45 within WSDOT's Olympic region. The existing structure at that location has been identified as a fish barrier by the Washington Department of Fish and Wildlife (WDFW) and WSDOT Environmental Services Office (ESO) (site identifier [ID] 991572) and has an estimated 7,175 linear feet (LF) of habitat gain.

Per the federal injunction, and in order of preference, fish passage should be achieved by (1) avoiding the necessity for the roadway to cross the stream, (2) use of a full-span bridge, or (3) use of the stream simulation methodology. WSDOT evaluated the crossing using the Unconfined Bridge Methodology. This methodology was chosen because the stream is considered unconfined according to a threshold of 3.0 for the floodplain utilization ratio. Although the Unconfined Bridge Methodology was used for the preliminary crossing design, no structure is recommended at this stage.

The crossing is located in Kitsap County, 1 mile northeast of Poulsbo, Washington, in WRIA 15. The highway runs in a north-south direction at this location and the confluence with the mainstem of Dogfish Creek is approximately 0.9 miles downstream. Northeast Dogfish Creek generally flows from northeast to southwest beginning approximately 1 mile upstream of the SR 307 crossing. A preliminary hydraulic design for another WSDOT crossing on SR 307 milepost 1.34 (ID 991999) is being developed concurrently (see Figure 1 for the vicinity map).

The proposed project will replace the existing corrugated steel, 110.4-foot-long, 48-inch-diameter culvert with a structure designed to accommodate a minimum hydraulic width of 25 feet. The proposed structure is designed to meet the requirements of the federal injunction using the unconfined bridge methodology as described in the 2013 WDFW *Water Crossing Design Guidelines* (WCDG) (Barnard et al. 2013). This design also meets the requirements of the WSDOT *Hydraulics Manual* (WSDOT 2022a).

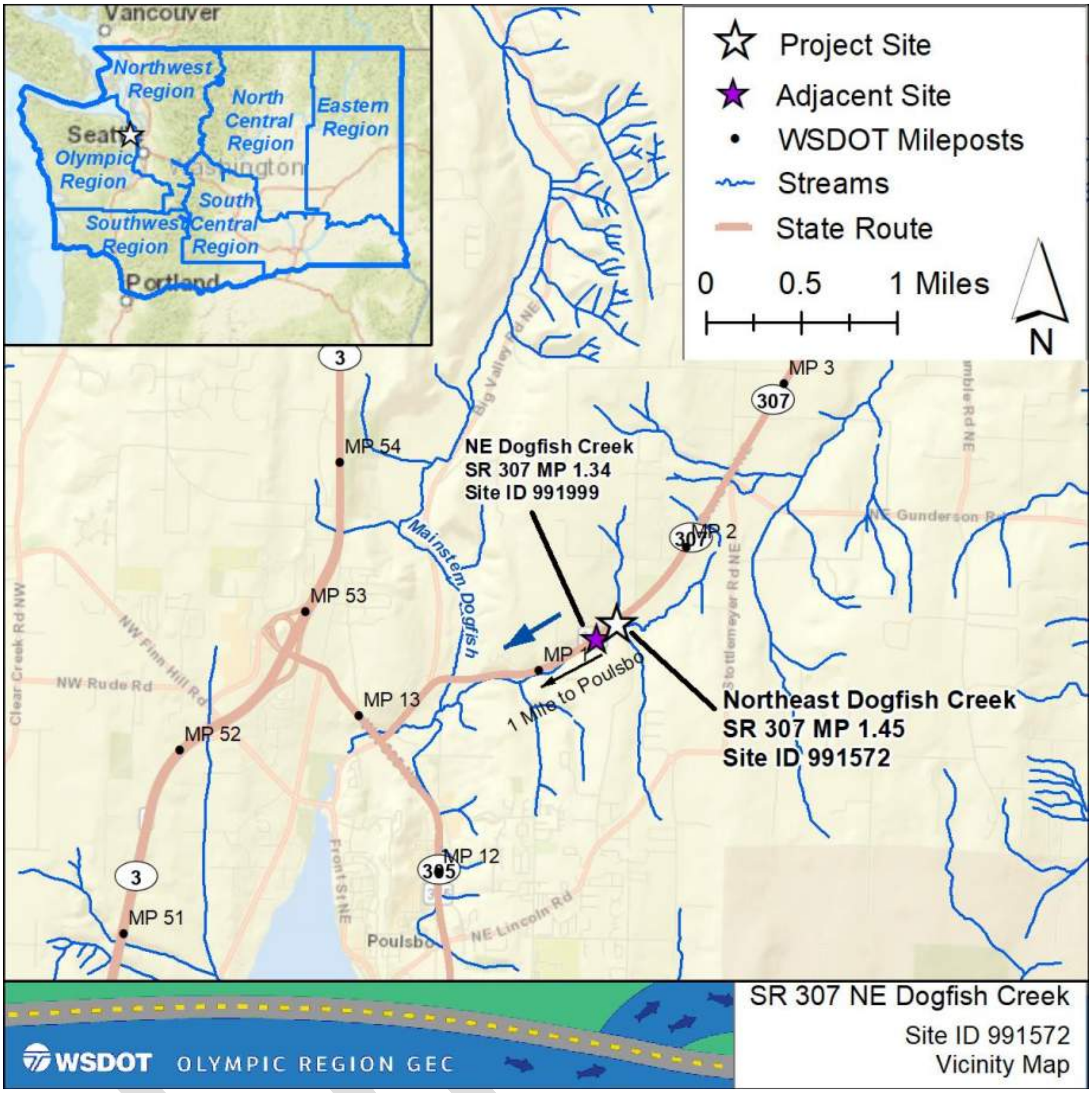

Figure 1. Vicinity map

2 Watershed and Site Assessment

The existing watershed was assessed in terms of land cover, geology, regulatory floodplains, fish presence, site observations, wildlife crossing priority, and geomorphology. This was performed using a site visit and desktop research with resources such as the United States Geological Survey (USGS), Federal Emergency Management Agency (FEMA), and WDFW, and past records like observations, maintenance, and fish passage evaluation.

2.1 Site Description

Crossing 991572 (the Project Crossing) is in Kitsap County in Water Resource Area (WRIA) 15 on SR 307 milepost 1.45. This crossing has been designated by WDFW as a partial fish passage barrier, with a passibility rating of 33 percent and 7,175 LF of habitat gain (WDFW 2021). The recorded reason for the barrier status is a water surface drop at the outlet of the culvert, which was documented during the 1999 barrier inventory survey. During site investigations for the preliminary hydraulic design no water surface drop was seen at the outlet (Section 2.6.2), although the existing culvert is undersized and is likely a barrier to fish passage regardless of hydraulic conditions at the outlet. Beyond being a barrier to migration, this culvert is likely to limit sediment and wood transport to downstream reaches leading to reduced habitat complexity below the crossing. The site is not designated as a Chronic Environmental Deficiency (CED) and does not have a known history of flooding. There are no maintenance or repair records for this crossing.

2.2 Watershed and Land Cover

The watershed which drains to the project area (the Project Basin) is 1,228 acres. This includes the Northeast Dogfish Mainstem basin (793 acres), South Tributary (213 acres) and North Tributary (218 acres) (Figure 2). The North Tributary joins Northeast Dogfish Creek immediately downstream of the Project Crossing and the flow it contributes is considered in the sizing and design of the stream channel. The Project Basin has a maximum elevation of 300 feet and a minimum elevation of 135 feet (North American Vertical Datum of 1988, NAVD88). The inlet elevation of the Project Crossing is 150.0 feet and the outlet elevation is 148.5 feet. The Project Basin and subbasin boundaries were delineated using Kitsap County OPSW 2018 LiDAR (DNR 2018).

Land cover in the basin (Figure 3) was summarized using the National Land Cover Database (NLCD 2019). The dominant land cover in the basin is forest, which covers 40.7 percent of the basin (Table 1). Herbaceous and shrubland combine to cover an additional 25.3 percent of the basin. The basin has 30.5 percent land cover which is considered Developed, ranging from Open Space to High Intensity development. Landcover in the basin is 9.0 percent planted/cultivated and 8.5 percent of the basin is wetlands. As the land cover classes give ranges of imperviousness, the percent impervious cover for the basin was calculated using the NLCD Urban Imperviousness dataset. The Northeast Dogfish Mainstem basin is 6 percent impervious; the South Tributary basin is 13 percent impervious, and the North Tributary basin is 8 percent impervious (Figure 4).

Approximately 800 feet downstream of the Project Crossing Northeast Dogfish Creek crosses SR 307 again. This downstream crossing, Crossing ID 991999, is also owned by WSDOT and is scheduled to be replaced. At the time of writing the Preliminary Hydraulic Design for that site is being developed.

Table 1. Land cover (NLCD 2019)

Land cover class	Basin coverage (percentage)
Developed	30.5
Barren	0.2
Forest	40.7
Shrubland	4.0
Herbaceous	7.1
Planted/Cultivated	9.0
Wetlands	8.5

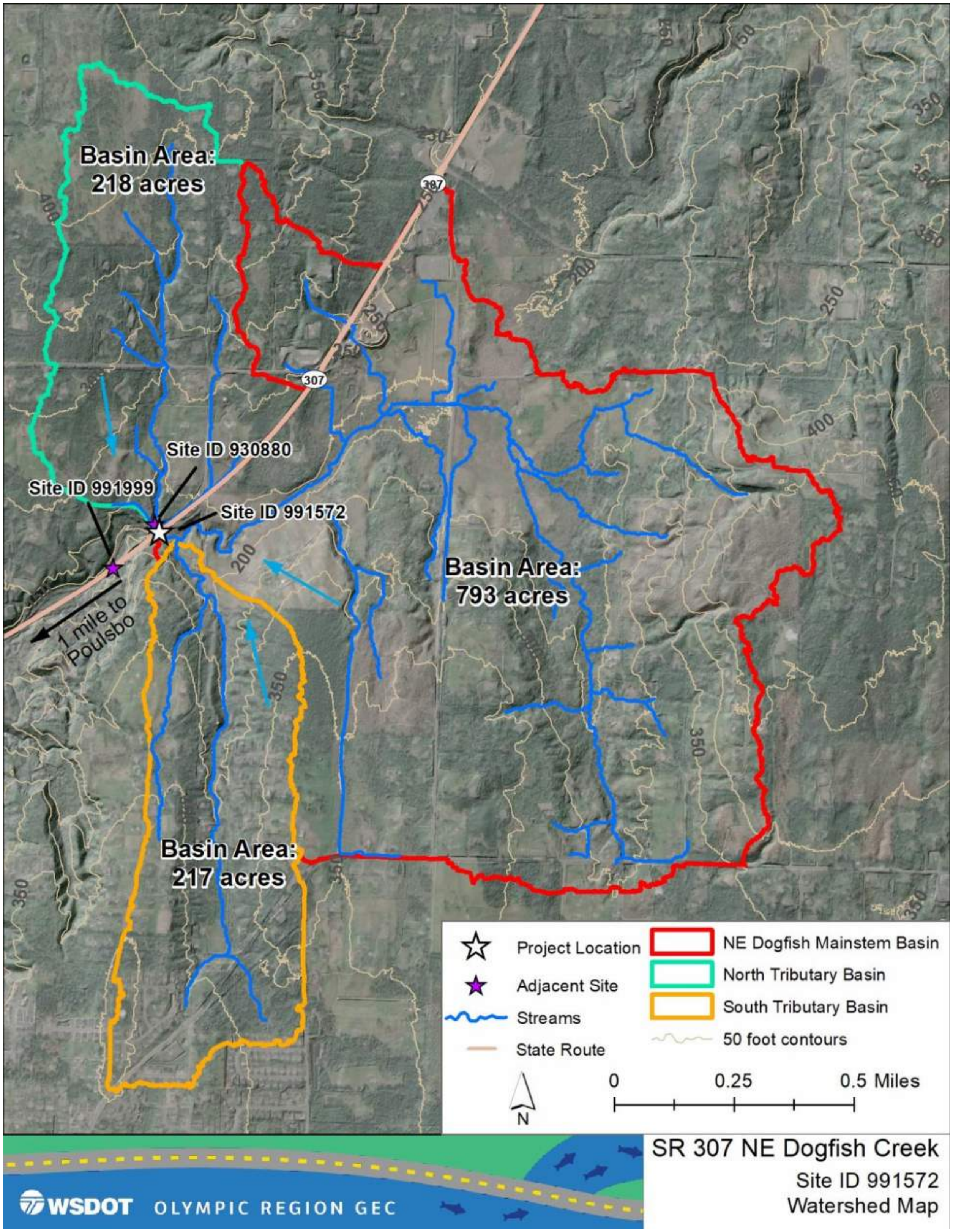

Figure 2. Watershed map

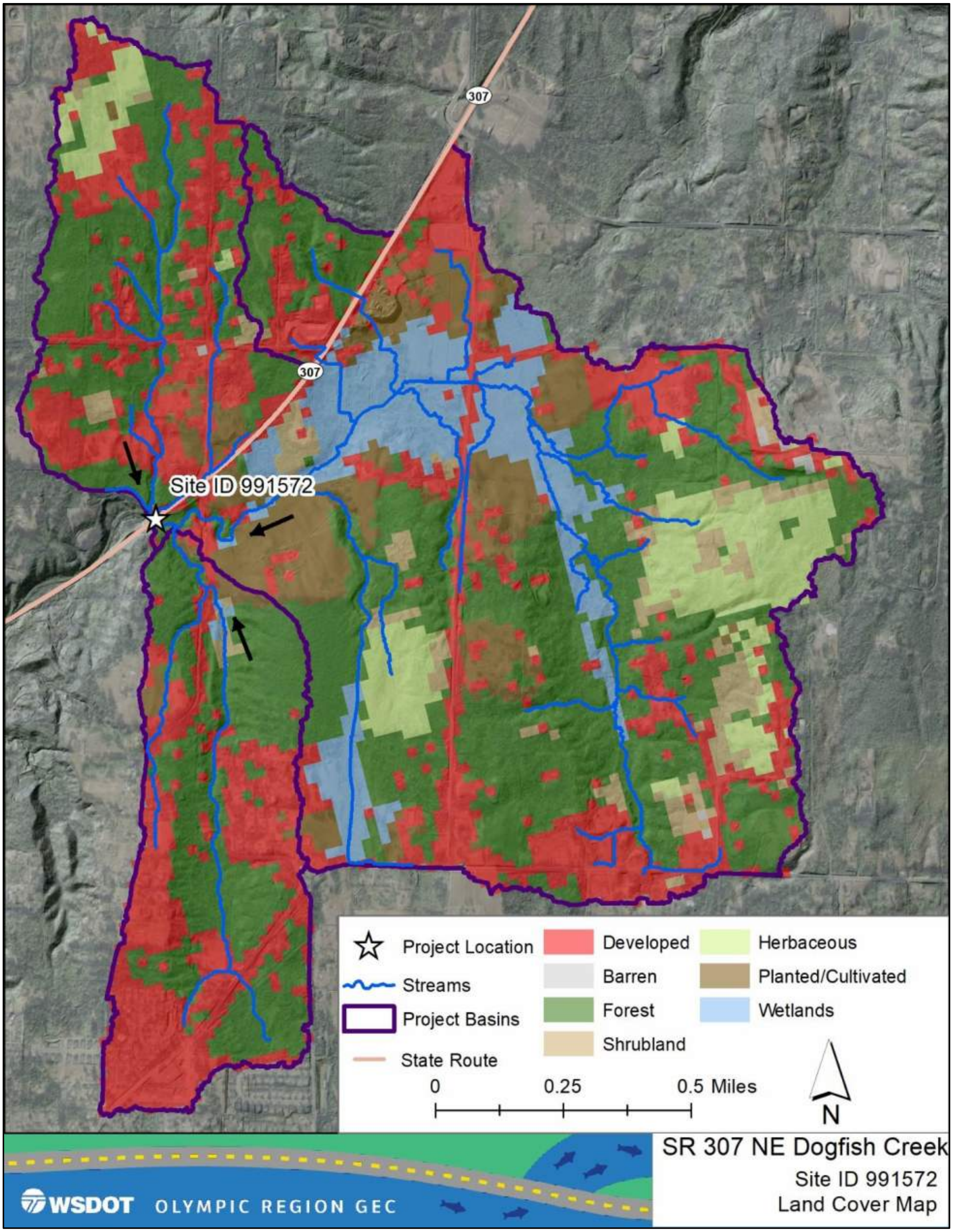

Figure 3. Land cover map (NLCD 2019)

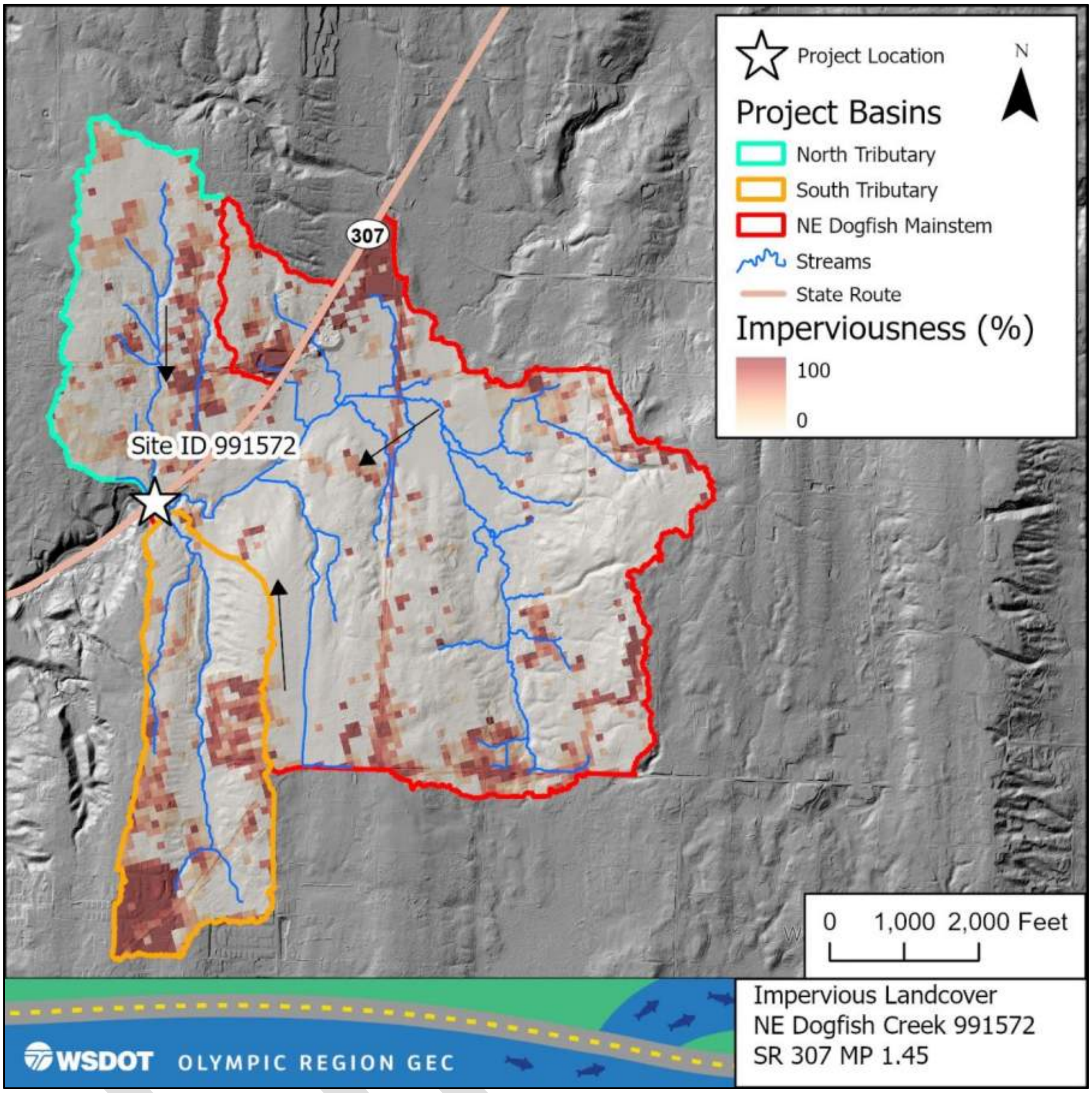

Figure 4. Impervious land cover from NLCD 2019 Imperviousness dataset

2.3 Geology and Soils

The Project Basin is located within the Puget Lowland, a low-lying area between the Cascade Range to the east and the Olympic Mountains to the west. The geology of the Puget Lowland reflects multiple periods of glacial advance and recession occurring throughout the Pleistocene epoch. Geology within the Project Basin was obtained from Department of Natural Resources (DNR) geologic mapping (Figure 5) (DNR 2018). At the time of writing no geotechnical information was available for this crossing.

The 1:100,000 scale geologic mapping shows Pleistocene continental glacial till (Qgt), Pleistocene continental glacial drift (Qgd), and Quaternary alluvium (Qa) are the dominant geologic units found within the Project Basin.

Pleistocene continental glacial till, also known as Vashon Till, underlies the majority of the Project Basin. The till deposits are found higher in the basin on hills, ridges, slopes, and generally away from surface flow paths. Till consists of a non-sorted mixture of mud, sand, pebbles, cobbles, and boulders. The till deposit is generally compact and often is referred to as hardpan, which has high resistance to surface erosion and landslide. When glacial till is not compacted into a hardpan it is often very silty. Fine sediments in the system are from non-compacted glacial till; however, the predominate characteristic is compacted glacial till which does not infiltrate well. The hydrology is modeled to account for this hardpan affect (Section 3).

Valley bottoms and wetlands in the Project Basin are composed of Pleistocene glacial drift or younger Quaternary alluvium deposits. The glacial drift deposits are a heterogenous patchwork of stratified and unstratified till, outwash, and ice-dammed-lake sediments. Glacial drift is often composed of sand to pebble gravel with minor silt. Quaternary alluvium deposits are well-rounded and well-sorted cobble gravel, pebbly sand and sandy silt. These two features are the dominant geologic units along the mainstem of Northeast Dogfish Creek. Much of the sediment seen near the Project Crossing was alluvial gravels, but some larger cobbles and small boulders were seen (Section 2.7.3). These are likely glacial drift deposits.

The soils in the Project Basin are predominantly sandy and gravelly loam (Figure 6). The majority of soil units in the Northeast Dogfish Creek Mainstem basin and the Southern Tributary basin fall into hydrologic group B/D. This indicates the soils have a naturally slow infiltration rate (group D) but have been artificially drained so they now have a moderate infiltration rate (group B). The soil units in the Northern Tributary Basin are predominantly in hydrologic group B, which indicates they have a moderate infiltration rate. Higher infiltration rates result in lower peak flows as rainfall is stored locally where it falls and translates to surface flow channels over a longer period of time. See Section 3 for the hydrologic analysis.

The slow infiltration rates indicated by the geological and soils data result in increased peak flows. The channel design is scaled so that these peak flows interact with the floodplain as seen in the existing and natural conditions models (see Section 0 for model results).

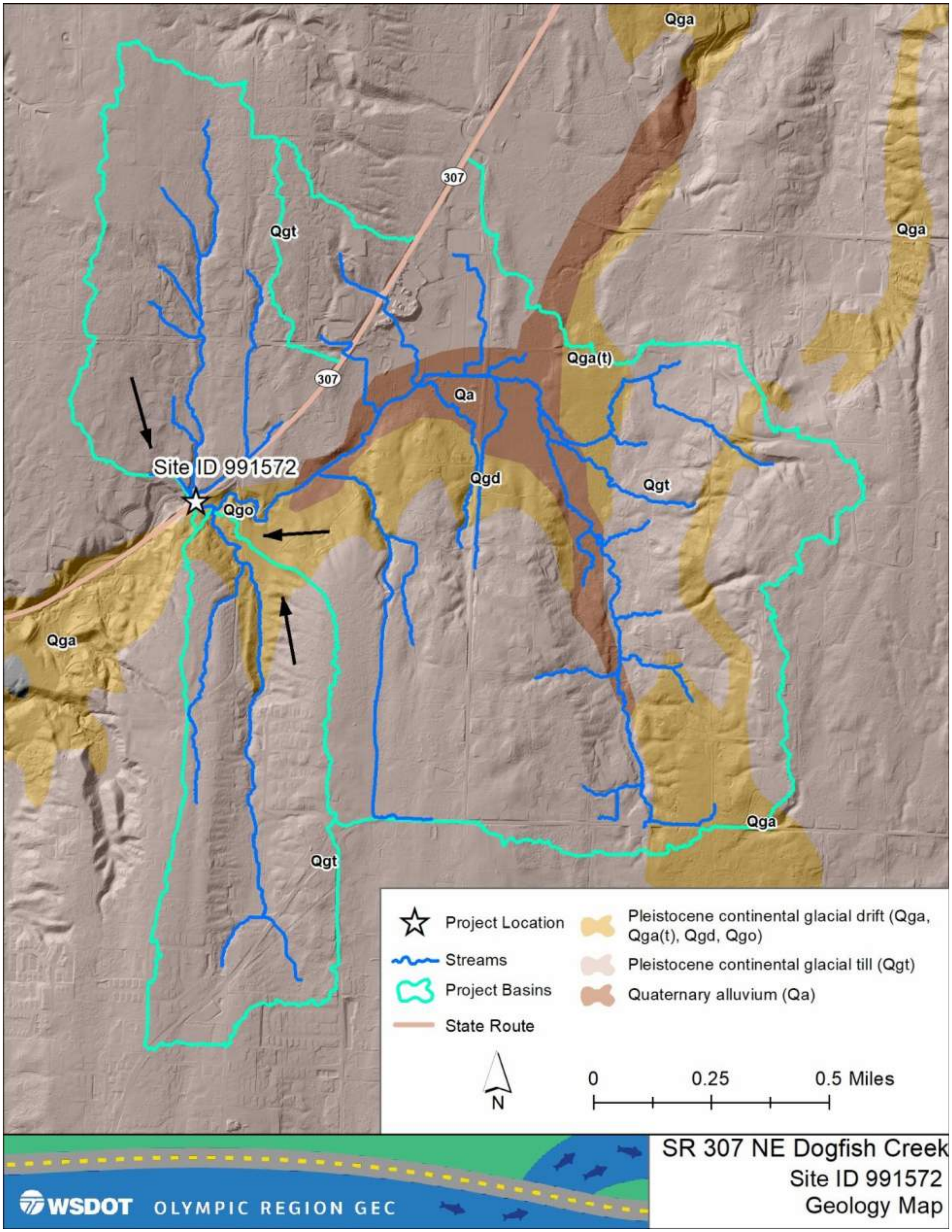

Figure 5. Geologic map

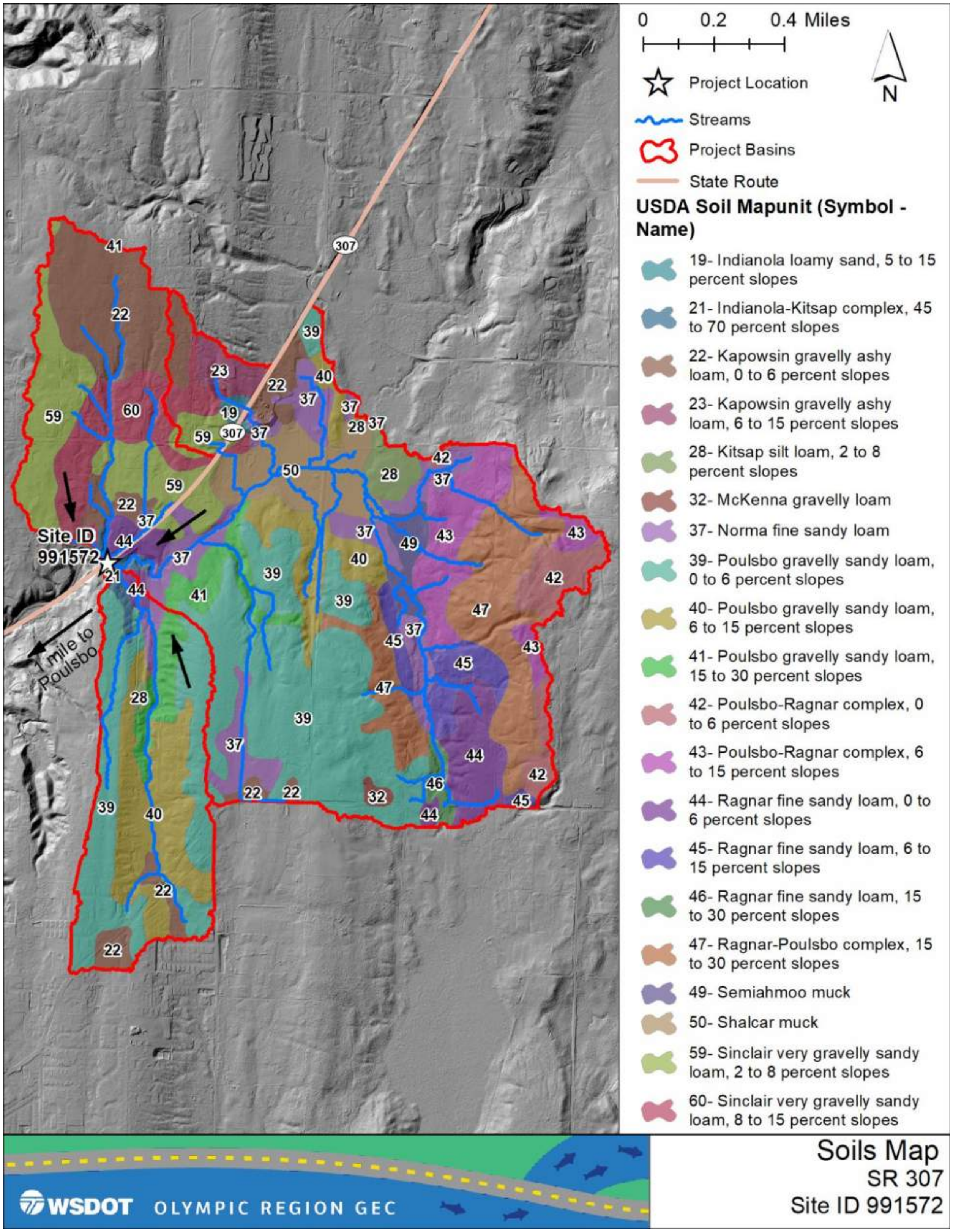

Figure 6. Soils map

2.4 Fish Presence in the Project Area

Multiple species of salmonids have been documented at the project site. According to the Statewide Integrated Fish Distribution (SWIFD) database, fall Chum salmon (*Oncorhynchus keta*), Coho salmon (*O. kisutch*), winter Steelhead (*O. mykiss*), and sea-run cutthroat (*O. clarkii*) all have a documented presence up to and above the project crossing (NWIFC and WDFW 2022). The Rapid Sample Full Survey (RSFS) identified these species and resident trout as potentially benefiting from the project (WDFW 2019, unpublished data). The presence of fall Chinook has been documented up to about 2,000 feet below the project crossing (NWIFC and WDFW 2022). A 'presumed' presence for resident trout is defined as a reach having no natural barriers downstream and using best biological judgement (McTeague and O'Connor 2006).

Puget Sound Coho are not listed under the Endangered Species Act (ESA). Puget Sound Fall Chinook, which were documented in the tributary below the site, are listed as threatened (NOAA Fisheries 2022). The Steelhead at the project site are part of the Puget Sound Distinct Population Segment, which are also listed as threatened (Cram 2018). Chum, which are documented at the project site, are part of the threatened Hood Canal Chum Summer-Run evolutionary significant unit (NOAA 1999). Fall Chinook in Northeast Dogfish Creek are part of the threatened Puget Sound Chinook Salmon evolutionary significant unit (NOAA 1999b). None of the resident or sea-run cutthroat trout Distinct Population Segments are listed under the Endangered Species Act (Connolly 2008).

Table 2. Native fish species potentially present within the project area

Species	Presence (presumed, modeled, or documented)	Data source	ESA listing
Coho	Documented	SWIFD and Barrier Inventory	Not listed
Steelhead	Documented	SWIFD and Barrier Inventory	Threatened
Chum	Documented	SWIFD and Barrier Inventory	Threatened
Fall Chinook	Documented (downstream)	SWIFD and Barrier Inventory	Threatened
Sea-run Cutthroat	Documented	SWIFD and Barrier Inventory	Not listed
Resident trout	Presumed	Barrier Inventory	Not listed

2.5 Wildlife Connectivity

Wildlife Connectivity will only be included in the FHD if Wildlife Connectivity is included as part of the project.

2.6 Site Assessment

Crossing WDFW ID 991572 (the Project Crossing) sits at the bottom of a steep forested ravine. Upstream of the Project Crossing the bottom of the ravine is 50 to 100 feet wide with a low bench outside of the bankfull channel. Immediately downstream of the Project Crossing the valley bottom narrows due to confinement from the road embankment. In this reach the channel runs parallel to SR 307 and becomes entrenched, with banks 4 to 5 feet tall. The surrounding residential properties add some complexity as directly downstream of the crossing there is another culvert (Site ID 930880) that runs under a private driveway. Approximately 900 feet downstream Northeast Dogfish Creek crosses SR 307 again at Site ID 991999. The Preliminary Hydraulic Design for this crossing is being developed by the WSDOT Olympic Region GEC in coordination with the design for Site ID 991572. The watershed delineation, hydrology, reference reach, and pebble counts are shared between the two sites.

2.6.1 Data Collection

Field data collection occurred over 5 different site visits. The date and purpose of the site visits was as follows:

- Feb 24, 2022 – Ground survey collected by WSDOT surveyors across several days in winter of 2021-2022, finalized on Feb 24th
- December 2, 2021 – Hydraulic site visit completed by PACE Engineers of the reach upstream of Site ID 991572
- December 9, 2021 – Hydraulic site visit completed by PACE Engineers of the reach downstream of Site ID 991572
- December 10, 2021 - Hydraulic site visit completed by GeoEngineers of the reaches upstream and downstream of Site ID 991999
- February 2, 2022 – Comanager concurrence site visit of Site ID 991572 and Site ID 991999 completed

Survey data was finalized on February 24, 2022, by WSDOT. The stream surveyed started approximately 1,100 feet downstream of the Project Crossing and includes crossing 991999, the approximately 800 feet between crossing 991999 and the project crossing 991572. The upstream end of the survey is approximately 200 feet above the project crossing. On December 2, 2021, engineers from PACE gathered three BFW measurements and two pebble counts in the reach upstream of Site ID 991572. On December 9, 2021, PACE Engineers revisited the site to investigate the reach downstream of Site ID 991572, collecting two additional BFW measurements. As the downstream reach for crossing 991572 is the same as the upstream reach of crossing 991999, it was decided a single shared reference reach for the two crossings was appropriate and would facilitate a continuity in design. GeoEngineers, the firm developing the Preliminary Hydraulic Design for crossing 991999, visited the site on December 10, 2021, and took three BFW measurements and three pebble counts.

During the concurrence meeting on February 2, 2022, comanagers (including WSDOT, WDFW, Suquamish Tribe, and Port Gamble S'Klallam Tribe) visited the site. The comanagers agreed a shared reference reach between the two sites would be appropriate. The comanagers took eight more bankfull width measurements in both the upstream and downstream reach. Ultimately, the concurrence bankfull width used only four bankfull width measurements collected in the reference reach. All locations of BFW and pebble count measurements can be seen in Figure 7. Only BFW measurements taken in the reference reach during the concurrence meeting are used in the average and numbered. Further discussion of BFW measurements and pebble counts can be found in 2.7.2 and 2.7.3, respectively. Refer to Appendix B for the field report.

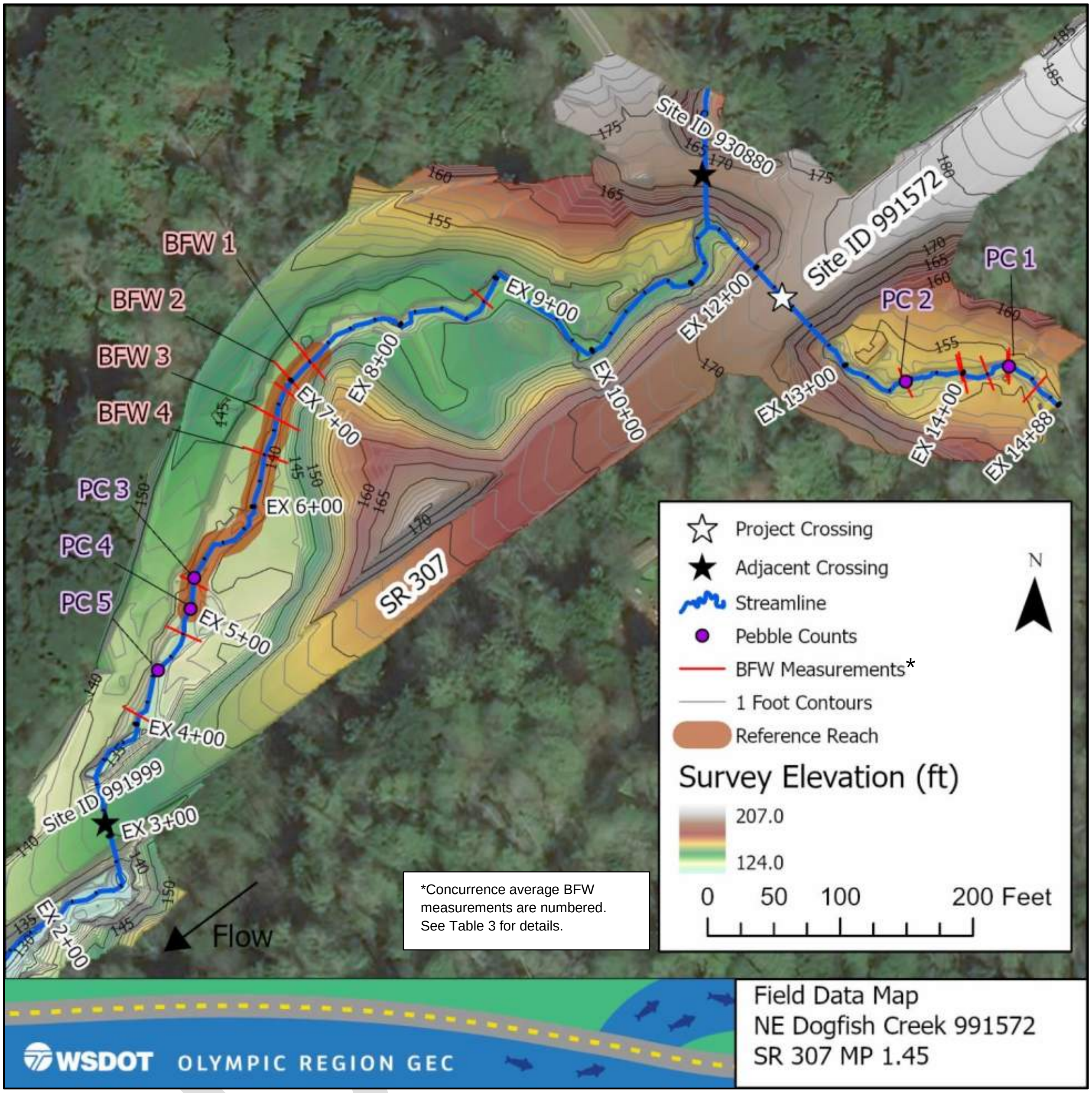

Figure 7. Reference reach, bankfull width, and pebble count locations

2.6.2 Existing Conditions

The existing crossing of SR 307 and Northeast Dogfish Creek is a 110.4-foot-long, 4-foot-diameter corrugated metal culvert and crosses SR 307 in a southeast to northwest direction. The surveyed culvert inverts show that the existing crossing has a 1.4 percent slope and that the crossing is nearly perpendicular to the road. There is approximately 18 feet of road fill over the structure. The banks are steep and there is woody material accumulated around the inlet (Figure 8). At the outlet there is a circular 0.9-foot-deep and 3-foot-long scour pool (Figure 9). In the existing condition the culvert invert is even with the creek bed, but the 1999 WDFW culvert assessment (WDFW 2021) reports there was a 1.1-foot water surface drop at the outlet (Figure 10). It is not known if natural aggradation or maintenance reversed the water surface drop at the outlet. The visible portions of the culvert show some signs of rust and dents, but the inside of the culvert is clear of debris and sediment (Figure 9). Several as-builts have been obtained detailing the various road repair projects on SR 307; however, no as-builts for the structure have been obtained. Bank slopes at the outlet are steep (2.5H:1V) due to the location of the outlet between the embankment fill from SR 307 and the private driveway containing crossing 930880.

Figure 8. Inlet of culvert

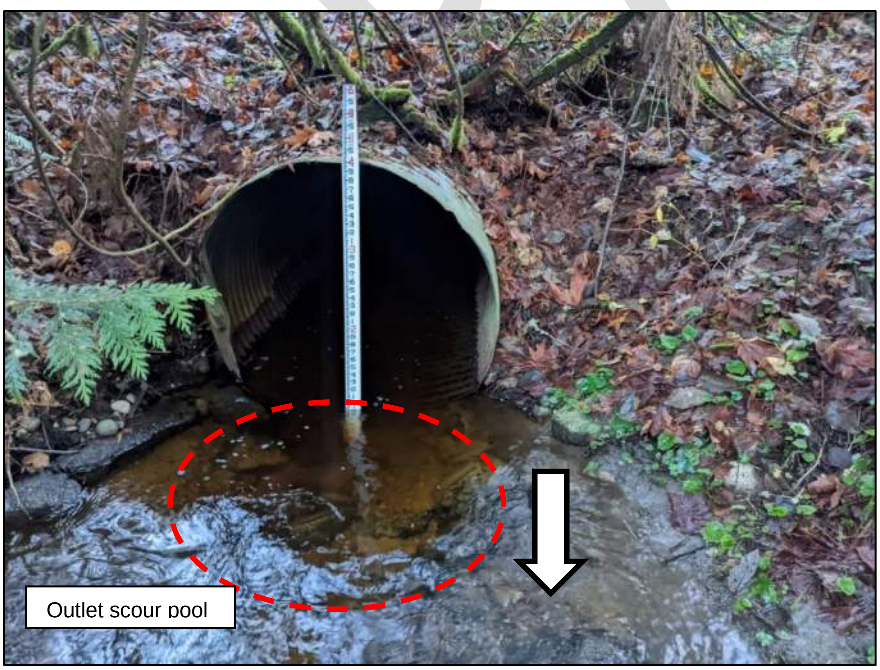

Figure 9. Outlet of culvert

Figure 10. 1999 culvert assessment showing a water surface drop at the outlet

Immediately upstream of the crossing the stream flows along the left valley wall. The left valley wall is steep with some evidence of undercutting. There is a road at the top of the left valley wall and the valley wall appears to be composed of native glacial drift. There is a shallow slope on the right bank leading to a floodplain bench (Figure 11). The creek moves away from the valley wall approximately 60 feet upstream of the inlet. There is a floodplain on both sides of the channel here, and occasional large woody material spanning from bank to bank (Figure 12). About 200 feet upstream of the crossing the South Tributary joins the mainstem (Figure 13).

Figure 11. Upstream of inlet, looking downstream

Figure 12. Channel spanning LWM, looking upstream

Figure 13. South Tributary joining the main flow, looking upstream

Immediately downstream of the crossing there is a confluence pool where the North Tributary meets the mainstem (Figure 14). The pool is approximately 8 feet long, 6 feet wide and 1.5 feet deep. Site ID 930880 is a corrected fish passage barrier which goes under a private driveway. Downstream of the confluence pool the channel is confined against the SR 307 road embankment (Figure 15, Figure 16). The left bank is formed by the road embankment and the right bank is vertical or undercut and is four to five feet tall. There are some larger cobbles and boulders in this reach, some of which appear to be angular and possibly related to road maintenance. No maintenance records were found, but during the concurrence meeting it was discussed that a design goal should be to move the stream alignment farther away from the toe of the SR 307 road fill to reduce future maintenance issues (Appendix B).

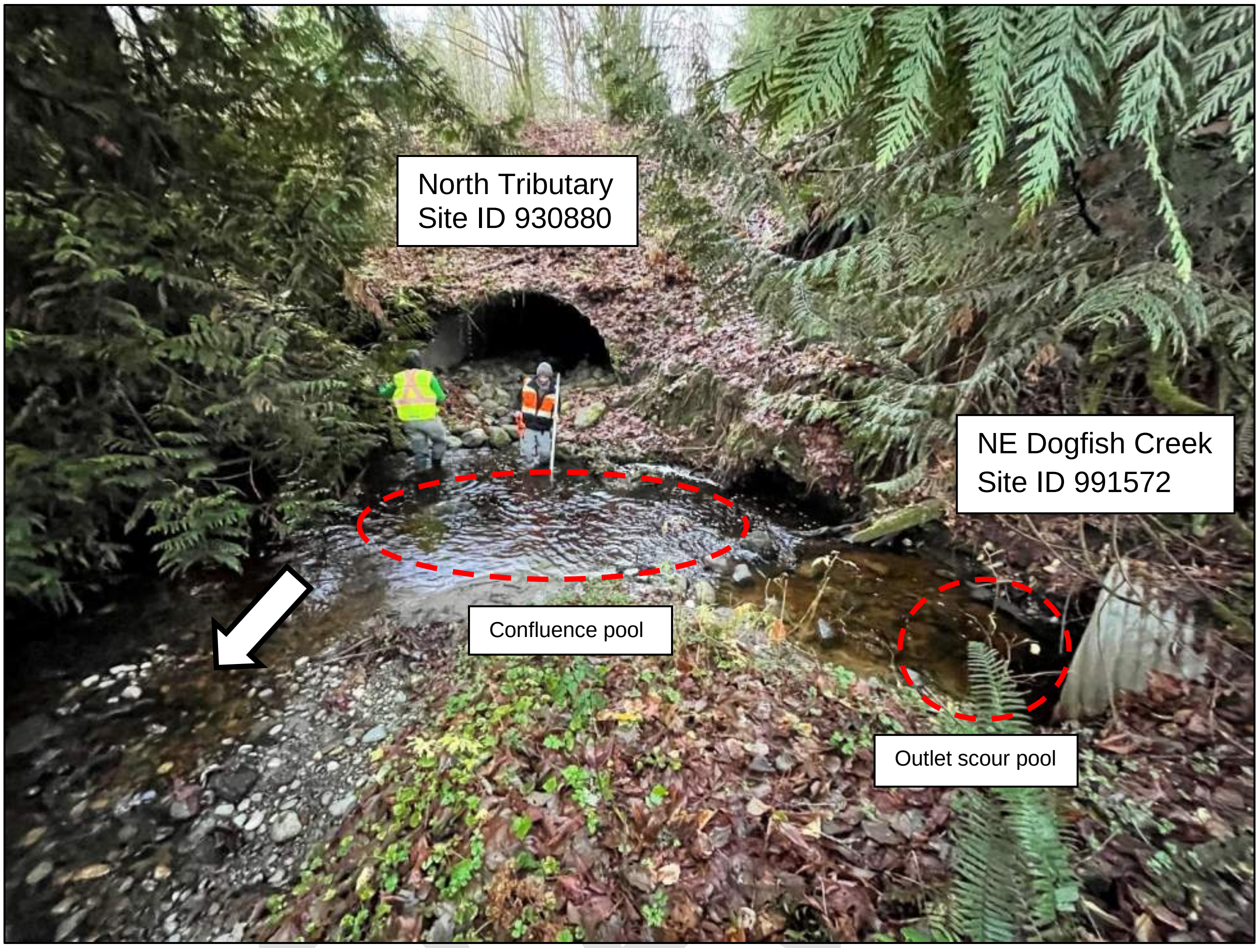

Figure 14. Outlet for crossing 991572 and crossing 930880, looking upstream

Figure 15. Immediately downstream of the confluence pool, looking downstream

Figure 16. Tall undercut bank in confined reach approximately 70 feet downstream of crossing

Approximately 140 feet downstream of the Project Crossing the stream channel makes a 90-degree turn away from the road embankment. At the bend there is a debris jam pool which is directing flow to the north and a backwater eddy (Figure 17). The left bank at the end of the backwater eddy is low and the topography indicates the channel may have historically continued straight along the road embankment (see Section 2.7.5 for more discussion). From there the downstream reach becomes less confined, with low vegetated benches on either side and pool riffle morphology (Figure 18).

Figure 17. Pool formed by LWM at 90-degree bend 140 feet downstream of crossing, looking downstream

Figure 18. Vegetated benches and channel spanning log

There is a small defined channel on the left (south) floodplain approximately 400 feet downstream of the crossing (Figure 19). The channel during the site visits was carrying seepage from the hillside. There were no indications this channel is actively connected to the flow of the main channel, such as alluvium.

Figure 19. Seepage channel on floodplain 400 feet downstream of crossing (EX 8+00)

Overall, there is good habitat in Northeast Dogfish Creek. The project crossing and SR 307 road embankment have likely increased confinement immediately upstream and downstream of the crossing. The existing crossing also appears to limit wood transport through the crossing (Section 2.6.4). Creating a fish-passable culvert will allow for better continuity of habitat forming processes in the vicinity of the project.

2.6.3 *Fish Habitat Character and Quality*

Overall, the habitat in Northeast Dogfish Creek to Dogfish Creek is of medium to high quality. There is natural recruitment of LWD, creating channel complexity and one log-jam pool at the time of the site visit. The gradient is relatively consistent upstream and downstream of the project crossing generally between 1.6 and 1.9 percent. The channel is forced pool-riffle throughout the project area. Spawning gravel can be found in the riffles of the less confined sections upstream of the crossing and downstream around the reference reach.

The most consistent high-quality spawning and rearing habitat is located upstream of the project crossing. Overhanging vegetation throughout this reach provides cover, habitat complexity, and likely increases the invertebrate supply to rearing juveniles. The gravel in this section generally has sands and silts filling the interstitial space, though clearing of fines during spawning may create suitable conditions for egg incubation. Undercut banks provide refuge for juvenile rearing. Closer to the project crossing LWD creates more pools separated by riffles and glides which offer more opportunities for spawning (Figure 20).

Just downstream of the project crossing a 2-foot-deep pool with an undercut bank at the confluence with an upstream tributary represents a potential adult holding spot and quality juvenile rearing habitat (Figure 14). The pool tail may provide spawning habitat. Downstream of the pool for approximately 150 feet the channel is incised and confined against the road bank of SR 307. This section has less habitat value, but boulders and coarse sediment create pocket water for juveniles and migrating adults. In addition, a debris jam with a pool has formed at the end of the constricted section which could provide good holding or rearing habitat (Figure 17). Downstream of this confinement the channel is more natural and offers better habitat value with riffles or glides between pools.

All fish species listed in Section 2.4 generally have similar habitat requirements. Chinook and chum don't overwinter in the freshwater and are more associated with main channel edge habitats. Coho overwinter for 1 or 2 years and are more likely to use off channel habitat. Steelhead and resident O. mykiss are generalists and will take advantage of all rearing habitats described.

Figure 20. Approximately 130 feet upstream of the crossing, looking upstream

2.6.4 Riparian Conditions, Large Wood, and Other Habitat Features

The riparian corridor of Northeast Dogfish Creek to Dogfish Creek within the vicinity of the project is largely undeveloped, with the exception of the SR 307 road embankment that the creek follows below the culvert. It is well vegetated with native tree and brush species. There was no noted beaver activity at the site during the site survey but it is possible there could be beaver activity near the crossing.

In the upstream reach, vegetation in the riparian corridor is composed of salmonberry (*Rubus spectabilis*), sword fern (*Polystichum munitum*), spreading wood fern (*Dryopteris expansa*), western red cedar (*Thuja plicata*) and big leaf maple (*Acer macrophyllum*). Western red cedars were found predominantly higher up on the slopes. Some trees along the bank are undercut while others have recruited into the stream and are habitat forcing features. No noxious weeds were noted.

There is LWM in the channel and on the banks upstream of the crossing. The crossing inlet is covered by several pieces of LWM, some of which appear to have fallen from the banks near the crossing or racked on the culvert during high flows. In the reach 0 to 50 feet upstream of the inlet there are several pieces of LWM on the banks or extending out to the toe of the bank, but there LWM has not forced any large scour pools. There is a large scour pool approximately 180 feet upstream of the crossing where a large spanning log forces water underneath it (Figure 21). Approximately 130 feet upstream of the crossing there is a 50-foot log lying along the toe of the bank, with another log spanning the channel and lying on top of the toe log. There are two more spanning logs at 175 feet upstream of the crossing and 195 feet upstream. At the confluence, approximately 230 feet upstream, there are several more pieces of the LWM (Figure 13). The amount of LWM in the channel and the number of trees along the bank suggests that the stream will continue to recruit wood into the future.

The downstream reach has a similar plant assemblage as the upstream reach, but the banks have a higher degree of evergreen trees. The overbank bench is dominated by cedars and other evergreens. Immediately downstream of the crossing until the first debris jam at 140-feet downstream there is sparse woody material, all of which is less than 1-foot in diameter. There are also exposed roots on the undercut banks. In this reach there are many pieces of woody material lying on the banks. At approximately 180 feet downstream of the crossing is a 2-foot-diameter spanning log which has racked several other 1- to 2-foot logs and other small organic debris (Figure 17). There is abundant 1- to 3-foot diameter wood which is spanning the channel approximately 270 feet downstream. Some of this appeared to have fallen recently, while other logs were moss covered and clearly had not moved in years. At 330 feet downstream there is a 1.5-foot diameter log with a root wad creating a scour pool (Figure 22).

There are occasional boulders which act as forcing features and create habitat. See Section 2.7.3 for a discussion.

Figure 21. Approximately 110 feet upstream a scour pool has formed under a log, looking upstream

Figure 22. Scour pool at 90-degree bend approximately 330 feet downstream of culvert

2.7 Geomorphology

Geomorphic information provided for this site includes selection of a reference reach, the geometry and cross-sections of the channel, and stability of the channel both vertically and laterally of Northeast Dogfish Creek.

2.7.1 Reference Reach Selection

The reference reach is located between STA EX 5+00 and STA EX 7+40, downstream of the Project Crossing (Figure 7). The reference reach is shared between the Site ID 991572 and Site ID 991999 to ensure continuity and compatibility between the designs (Figure 23). The substrate in the reference reach is primarily gravels, with some sand and cobble. The channel is forced pool-riffle and wood is the primary forcing structure (Figure 18). Pools in the reference reach are relatively shallow and measure less than 1-foot deep.

This reference reach was chosen because it is outside of the influence of the Project Crossing and Site ID 991999, has no abrupt changes in slope or sediment size, and has a similar slope to the project site. In addition, the reference reach represents the best available example of functioning natural channel processes and fish habitat. The reach upstream of the Project Crossing is influenced by backwater from the crossing. The reach immediately downstream of the Project Crossing runs along the toe of the SR 307 road fill and is highly confined which is atypical for NE Dogfish Creek and likely an artificial condition. There are no abrupt changes of slope or sediment size within the reference reach. The slope of the reference reach is 1.7percent. The downstream end of the reference reach is more unconfined than the upstream end due to a natural widening of the valley and floodplain.

Figure 23. Reference reach, collecting BFW 3, looking upstream

2.7.2 Channel Geometry

The channel planform has forced bends at wood jams which cause the channel to migrate from one side of the valley floor to the other (see Section 4.1.1 for further discussion on meandering). The valley bottom ranges from approximately 50 to 100 feet wide, while the channel is approximately 9 to 14 feet wide. The concurrence average bankfull width is 12.4 feet (Table 3). The design BFW was determined to be 12.0 to accommodate future stream widening. Bankfull was measured both upstream and downstream of the crossing, but during the concurrence meeting it was decided the bankfull average should use only the four bankfull width measurements taken in the reference reach on February 2, 2022. This provides continuity in design with Site ID 991999 and provides a factor of safety for the Project Crossing as the bankfull width measurements were taken in a reach which includes flow coming from the North Tributary. The North Tributary confluence is just downstream of the Project Crossing, so flows from the North Tributary do not go through the Project Crossing structure (Figure 7). The concurrence bankfull width rounded to the nearest foot is the design bankfull width which was used to create the proposed channel shape (Figure 36 and Section 4.1.1).

Table 3. Bankfull width measurements

BFW number	Location (STA)	Bankfull Width (ft)	Used in average	Concurrence notes
-	EX 13+75	17.0	No	Collected by PACE, December 2021. Stakeholder removed on 2/2/2022
-	EX 14+00	8.4	No	Collected by PACE, December 2021. Stakeholder removed on 2/2/2022
-	EX 14+40	9.9	No	Collected by PACE, December 2021. Stakeholder removed on 2/2/2022
-	EX 14+00	9	No	Collected by stakeholders 2/2/22, not used in average
-	EX 14+18	11	No	Collected by stakeholders 2/2/22, not used in average
-	EX 14+40	10	No	Collected by stakeholders 2/2/22, not used in average
-	EX 14+70	11	No	Collected by stakeholders 2/2/22, not used in average
-	EX 8+80	9.0	No	Collected by PACE, December 2021. Stakeholder removed on 2/2/2022
-	EX 6+90	11.0	No	Collected by PACE, December 2021. Stakeholder removed on 2/2/2022
-	EX 5+20	10.6	No	Collected by GeoEngineers, December 2021. Stakeholder removed on 2/2/2022
-	EX 4+80	11.9	No	Collected by GeoEngineers, December 2021. Stakeholder removed on 2/2/2022
-	EX 4+10	10.2	No	Collected by GeoEngineers, December 2021. Stakeholder removed on 2/2/2022
1	EX 7+20	14	Yes	Stakeholder added on 2/2/22
2	EX 7+00	10	Yes	Stakeholder added on 2/2/22
3	EX 6+70	13.5	Yes	Stakeholder added on 2/2/22
4	EX 6+20	12	Yes	Stakeholder added on 2/2/22
	Concurrence Average BFW	**12.4**		
	Design BFW	**12.0**		

Upstream of the Project Crossing bankfull width measurements ranged from 8.4 to 11 feet, with one measurement of 17 feet (Figure 25). During the comanager meeting it was decided only the measurements from the reference reach should be considered, and measurements from upstream of the crossing were not averaged into the concurrence average bankfull width. Measurements downstream of the crossing ranged from 9 to 14 feet (Figure 26). The concurrence bankfull width rounded to the nearest foot will be used to size the design bankfull channel. Typically bankfull widths are rounded up to the nearest foot, but given the crossing has a smaller drainage area than the crossing, the concurrence bankfull width was rounded down to get the design bankfull width. Model results indicate there is a good fit between the bankfull flow and the proposed channel shape (Section 5.2).

The slope of the reference reach and the slope of the upstream reach will be compared to the crossing design to calculate a slope ratio. Within the reference reach the slope is 1.7 percent and there are no slope breaks. This slope will be used as comparison for the design. The channel cross-sections taken within the reference reach show bank heights (relative to the thalweg) are 1.3 to 3.6 feet tall (Figure 24). Banks are vegetated and soils are composed of unconsolidated silts, sands and gravels. At the upstream end (STA EX 7+20) there is a hillside which forms one bank, and there is a 10- to 20-foot-wide bench approximately 3 to 4 feet above the channel on the other bank. Banks are nearly vertical in this area. At the downstream end of the reference reach (STA EX 5+40) banks have a 2H:1V slope to meet a 30- to 50-foot-wide floodplain which is approximately 2 feet above the thalweg.

The width to depth ratio is a metric that indicates the shape of the channel. It is calculated as the bankfull width divided by the bankfull depth. Larger values indicate a wide shallow channel and smaller numbers indicate the channel is narrow and deep. The width to depth ratio varies throughout the project, but generally is in the 4 to 10 range. The channel is quite narrow and deep immediately downstream of the Project Crossing (values <4). Immediately upstream of the Project Crossing and downstream of the Project Crossing in the reference reach the channel is banks are shallower. Width to depth ratios in the reference reach are between 6 and 10. Modeling results show both the 2- and 100-year flows are well connected to the floodplain (see Section 2.7.2.1).

The confined channel downstream of the Project Crossing has tall undercut banks and the 1999 culvert assessment report indicates the culvert had a hydraulic drop at the outlet, indicating the channel is in Stage 3, degradation, or Stage 4, degradation and widening (Cluer and Thorne 2014). It is not known why local aggregation happened at the culvert outlet (Figure 14), but this possibly due to local bank erosion or maintenance activity. Outside of the aggradation noted directly at the outlet, there is no indication the reach is aggrading. Upstream of the crossing and in the reference reach there is a single thread channel with a wide and well-connected floodplain, indicating the channel is in Stage 1 (sinuous single thread). There is a knickpoint approximately 1500 feet downstream of the Project Crossing. If it migrates upstream to the Project Crossing the entire reach would be forced into Stage 3, degradation.

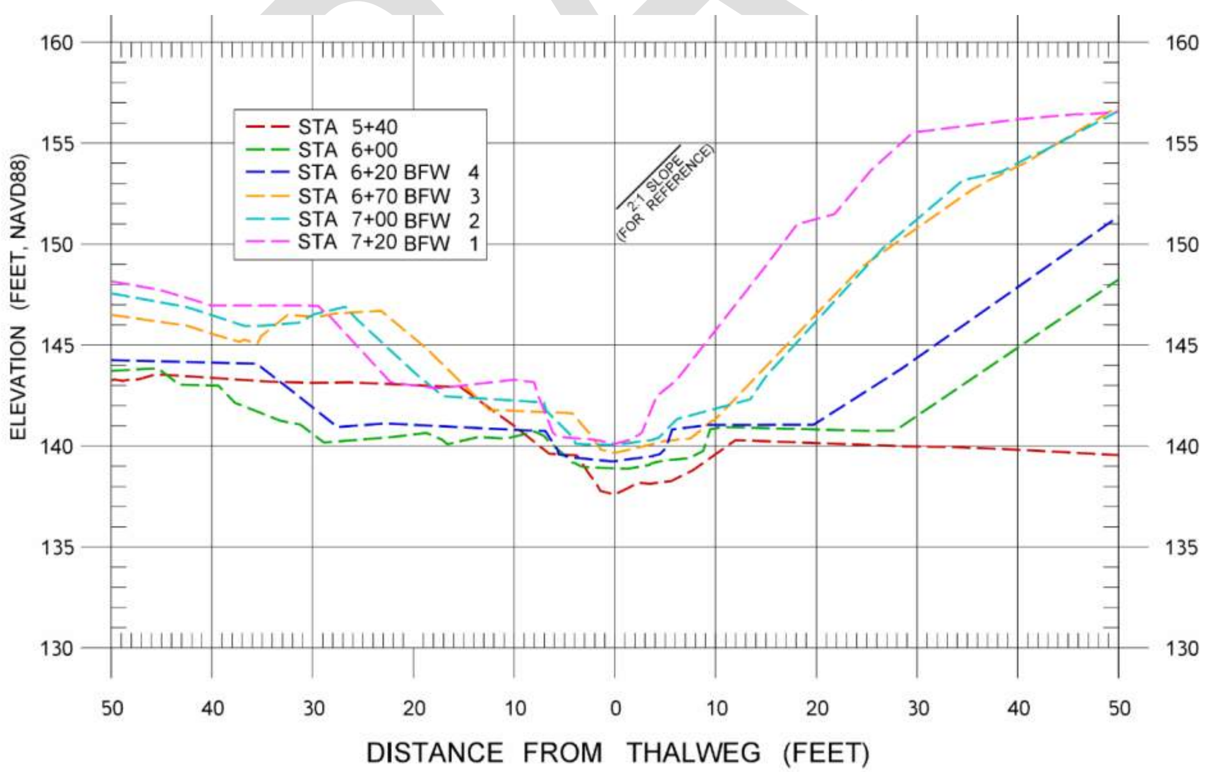

Figure 24. Existing cross-section examples, cross sections looking downstream

Figure 25. Measuring the channel geometry upstream of the crossing (9.0 feet, STA EX 14+00). Dashed lines represent the approximate location of bankfull identified in the field.

Figure 26. Measuring the channel geometry in the reference reach (13.5 feet, BFW 3, STA EX 6+20). Dashed lines represent the approximate location of bankfull identified in the field.

2.7.2.1 Floodplain Utilization Ratio

The floodplain utilization ratio (FUR) is a ratio between the flood-prone width (FPW) and the bankfull width. For this crossing, the modeled width of the 100-year flow was used as the FPW. The FPW was compared to the concurrence bankfull width average. Due to the undersized existing culvert there is significant backwater upstream of the crossing in the existing conditions model at the 100-year flow (Section 5.2). The FUR was measured using the natural conditions model (Section 5.3) in which the existing road fill has been removed entirely. Removing the road also removes any backwater impacts. FUR measurements were taken every 50 feet outside of the regraded area. The FUR upstream of the crossing in the natural conditions model varied between 2.7 and 3.9 (Table 4). Downstream of the crossing the FUR varied between 1.8 and 6.2, with the widest measurements being taken in the reference reach. The average FUR is 3.5, therefore the Project Crossing is unconfined.

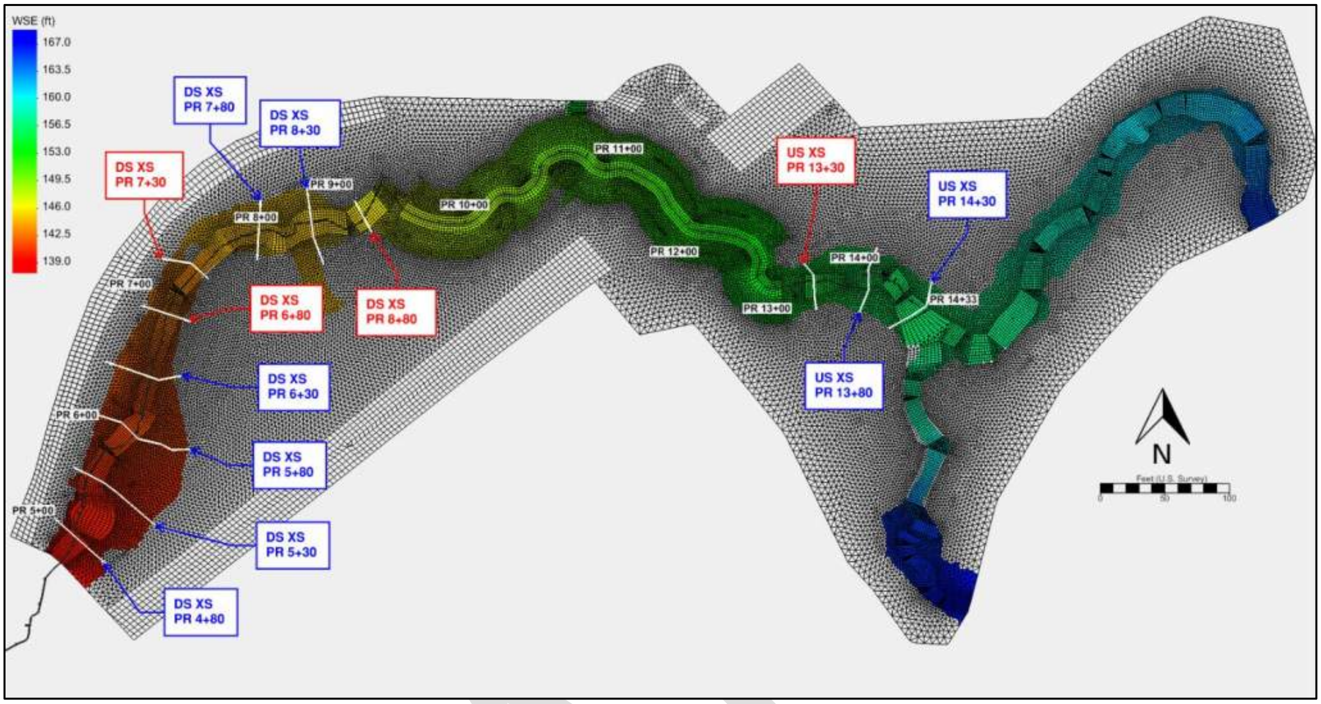

Figure 27. FUR locations overlaying natural conditions model 100-year flow extent. Blue and red callouts indicated unconfined and confined, respectively.

Table 4. FUR determination

Station	FPW (ft)	FUR	Confined/unconfined	Included in average FUR determination
US PR 14+30	48.2	3.9	Unconfined	Yes
US PR 13+80	47.3	3.8	Unconfined	Yes
US PR 13+30	33.8	2.7	Confined	Yes
DS PR 8+80	22.7	1.8	Confined	Yes
DS PR 8+30	45.0	3.6	Unconfined	Yes
DS PR 7+80	36.9	3.0	Unconfined	Yes
DS PR 7+30 (reference reach)	24.1	1.9	Confined	Yes
DS PR 6+80 (reference reach)	27.2	2.2	Confined	Yes
DS PR 6+30 (reference reach)	46.6	3.8	Unconfined	Yes
DS PR 5+80 (reference reach)	76.6	6.2	Unconfined	Yes
DS PR 5+30 (reference reach)	66.6	5.4	Unconfined	Yes
DS PR 4+80 (reference reach)	39.9	3.2	Unconfined	Yes
Average	**42.9**	**3.5**	**Unconfined**	

2.7.3 Sediment

At this crossing a total of five pebble counts were collected. PC 1 and PC 2 were collected in the upstream reach and PC 3, PC 4, and PC 5 were collected in the downstream reach (Figure 7). Sediment throughout the project reach was relatively uniform; therefore, all five pebble counts were averaged to inform project design. At each location the Wolman pebble count method was used. Pebble counts PC 1 and PC 2 were collected by PACE engineers at STA EX 13+60 and EX 14+40, respectively. As part of the shared design with Site ID 991999, pebble counts PC 3, PC 4, and PC 5 were collected by GeoEngineers at STA EX 5+20, EX 5+00 and EX 4+40 (Table 5). Two pebble counts collected upstream of the Project Crossing had a D_{50} of 0.5 and 0.7 inches, respectively (Figure 28). Downstream of the Project Crossing the sediment was slightly larger with a D_{50} of 1.0, 0.8 and 0.9 inches, respectively. All five pebble counts were averaged to inform the project design.

Table 5. Sediment properties near the project crossing

Particle size	PC1 diameter (in)	PC2 diameter (in)	PC3 diameter (in)	PC4 diameter (in)	PC5 diameter (in)	Average diameter for design (in)
Included in average?	Yes	Yes	Yes	Yes	Yes	
D_{16}	0.1	0.2	0.5	0.4	0.4	0.3
D_{50}	0.7	0.5	1.0	0.8	1.0	0.8
D_{84}	1.5	1.7	1.8	1.7	1.9	1.7
D_{95}	2.1	6.0	2.5	2.3	2.5	3.1
D_{100}	3.5	10.1	5.0	3.5	3.5	5.1

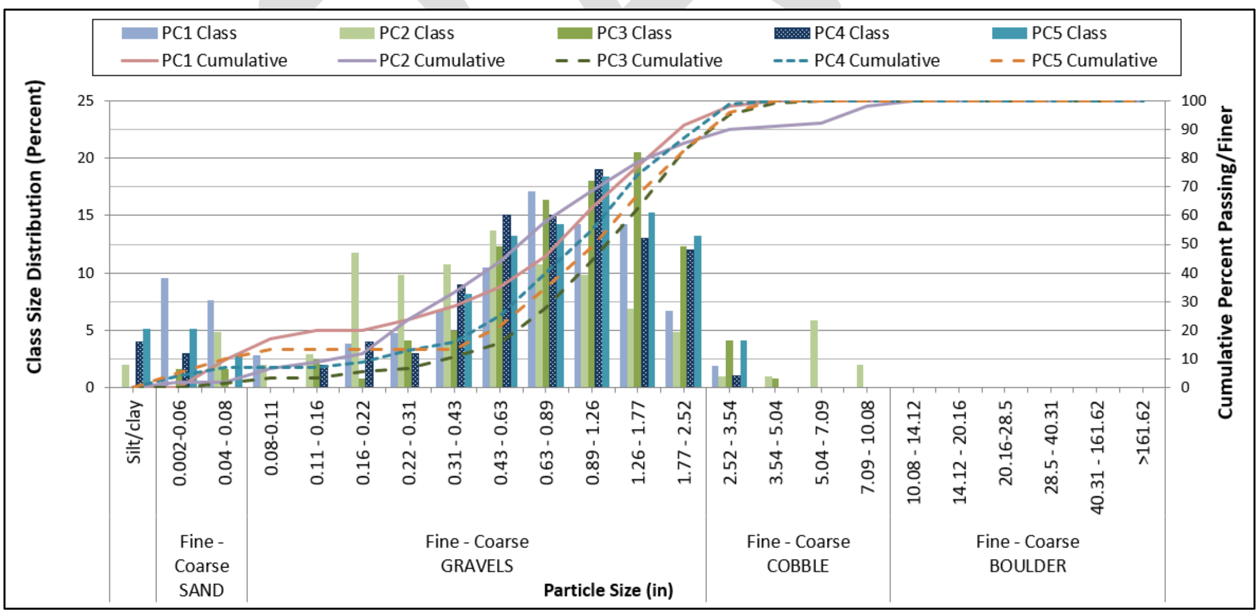

Figure 28. Sediment size distribution upstream and downstream

Upstream of the project crossing the streambed is generally composed of gravels (Figure 29). Near the bank edges and in low velocity areas sands and silts were observed. No boulders were captured in the pebble count, but occasional boulders were observed. There were two small (11-15 inch) boulders seen in the upstream reach. The silts observed are likely glacial till sediments and the larger rounded clasts seen are likely glacial outwash (Section 2.4). No armoring was observed in the reach upstream of the crossing.

Figure 29. Photo of gravels and small boulder taken upstream of the Project Crossing

Immediately downstream of the project crossing the channel bends sharply to the southwest and runs along the toe of the SR 307 road fill (Figure 7, Figure 16). The channel becomes more confined in this reach. Coarser sediments, large cobbles, and small boulders (12 to 18 inch) were observed here. The boulders had moss on the top of them so are either immobile or only mobilized infrequently. There were no maintenance records found for this area, but angular rocks and what appeared to be pieces of pavement were observed (Figure 30). This reach is likely artificially over-coarsened with a surface armor layer, and no pebble counts were collected here.

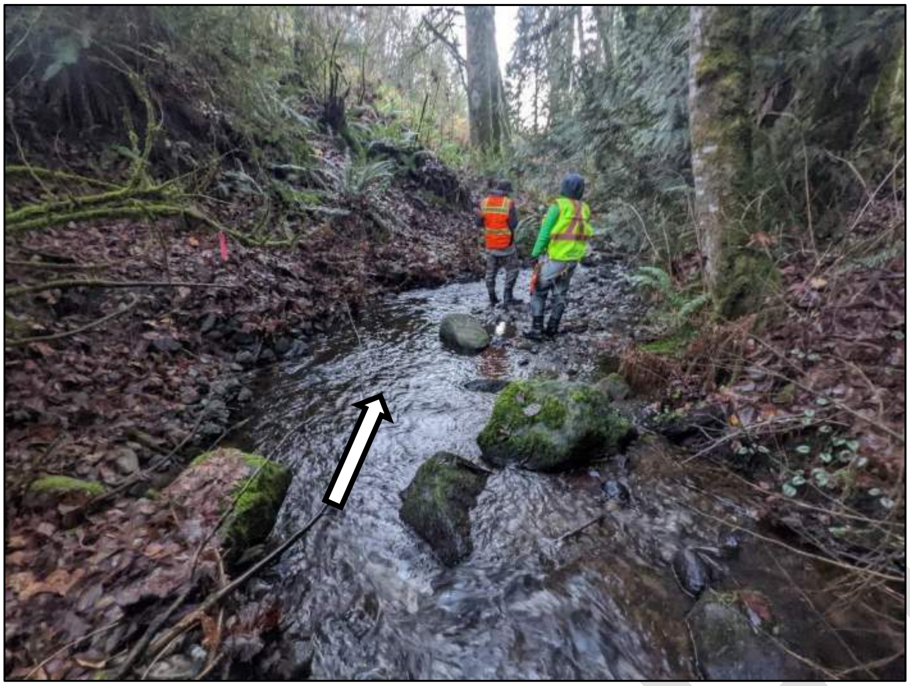

Figure 30. Downstream of the Project Crossing where the channel runs along the toe of SR 307, STA EX 10+50

Once the stream bends away from SR 307 at STA EX 10+00, the streambed sediment returns to predominantly gravels and cobbles (Figure 31). Pockets of finer sediment were observed. No boulders were observed in this reach.

Figure 31. Streambed sediment in the reference reach

2.7.4 *Vertical Channel Stability*

Vertical channel stability was assessed using a profile created from Kitsap County OPSW 2018 LiDAR (DNR 2018) and the ground survey completed by WSDOT. A LiDAR profile at the watershed scale reveals the slopes upstream and downstream of the project crossing are consistently between 1.5 and 2 percent within 2,000 feet of the crossing (Figure 32). The ground survey which has been placed over the LiDAR profile appears slightly offset at the upstream end because the thalweg distance captured in the ground survey is slightly longer than the thalweg captured in the LiDAR.

There are two slope breaks of note in the watershed scale longitudinal profile. Approximately 3,000 feet upstream of the crossing (STA 40+00 to 60+00) there is a large marshy area and the slope reduces to less than 0.5 percent. The marsh, which is mapped as wetland in the NLCD 2019 data, is approximately 2,000 feet long (Figure 3). An equilibrium slope with a constant slope of 1.8 percent fits the 7,000 foot reach downstream of the marsh, and this reach of constant slope is assumed to be the current watershed base level. Approximately 1,500 feet downstream of the Project Crossing and 300 feet downstream of Site ID 991999 (STA -0+50), there is a 3-to-4-foot knickpoint which may become the future base level control point. The drop was not included as part of the ground survey, but analysis of the LiDAR data confirms it is approximately 3 feet tall (Section 7.2). There is a jam of wood and live roots at the knickpoint, indicating it has not migrated in several years (Figure 33).

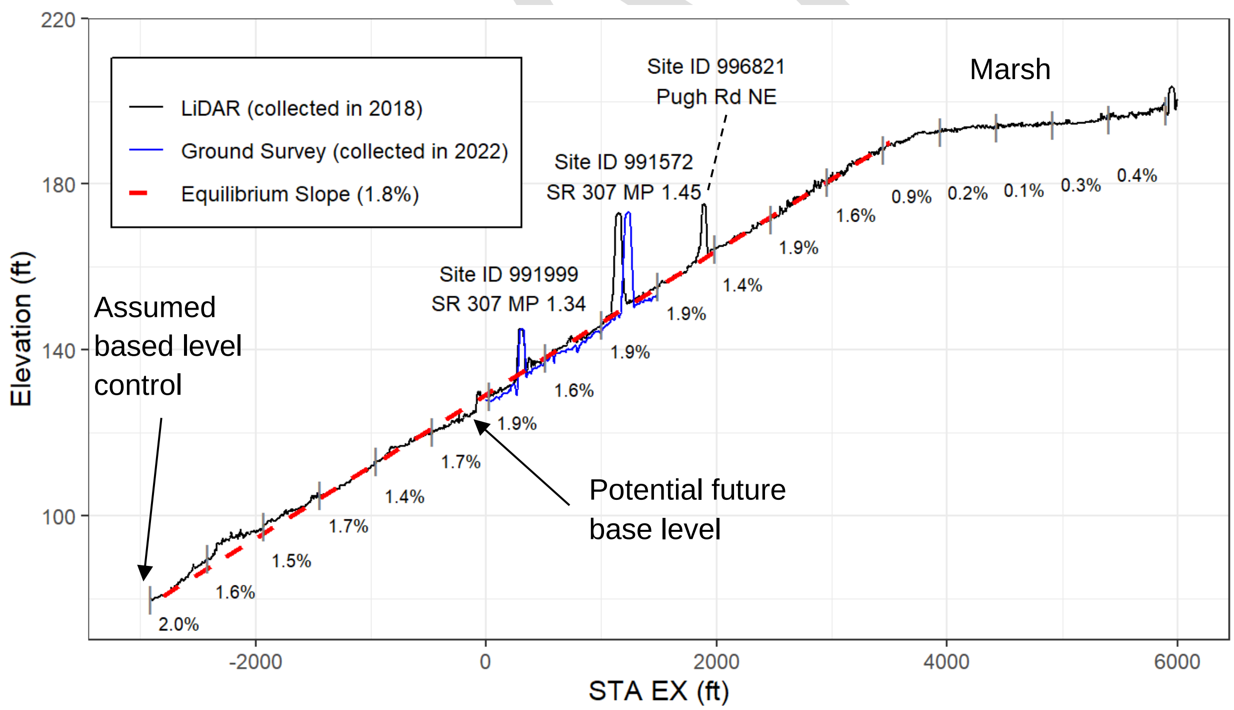

Figure 32. Watershed-scale longitudinal profile showing the ground survey and LiDAR

Figure 33. Jam of large wood, live roots and sediment forming a 3-to-4-foot drop just downstream of the ground survey limits, STA EX -0+50

The ground survey confirms the slopes seen in the LiDAR profile (Figure 34). Slopes were measured over approximately 200-foot intervals. The slope downstream of the Project Crossing is a consistent 1.9 percent slope for 400 feet. The slope of the reference reach is 1.7 percent. Upstream of the crossing the slope is 1.6 percent. There are no indications of vertical instability throughout the surveyed profile and there is no existing grade control other than the existing culvert.

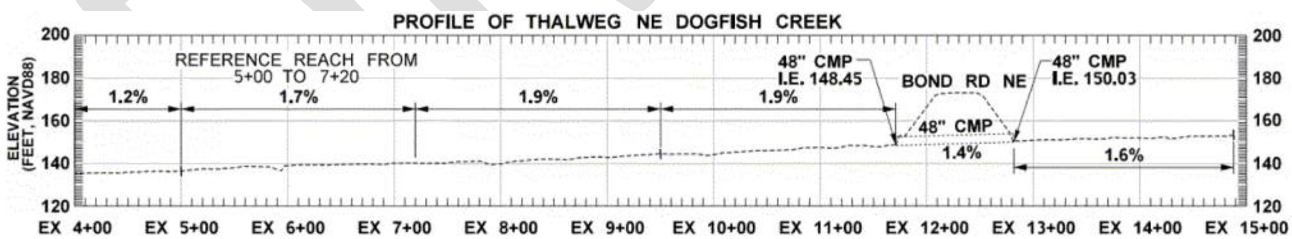

Figure 34. Longitudinal profile from ground survey data

The basin appears to supply an adequate amount of sediment to the project location, with clean alluvial gravels being the dominant substrate. Small pockets of sands and silts were observed as were occasional boulders. The low slope marsh upstream does not appear to create a supply limited condition at the project site. The stream within the vicinity of the Project Crossing is believed to be vertically stable with a small probability of significant and chronic aggregation or degradation.

The general characterization of the stream is that it is unconfined with active floodplains that are regularly inundated (Section 2.7.2.1). However, the reach immediately downstream of the Project Crossing has tall banks and is confined against the toe of the SR 307 road fill (Figure 16). It is possible the stream historically was unconfined in this reach, but over time it has become confined. The timeframe for this incision is not known, but potentially it was before the current crossing was constructed. The WDFW barrier inventory report (WDFW 2021) indicates there was a water surface drop at the outlet of the Project Crossing in the 1999 survey (Figure 10). Aggradation has occurred since 1999 and there is no longer a water surface drop. It is not known if this aggregation happened naturally or from maintenance activities. The water surface drop documented in 1999 suggests one to two feet of degradation near the culvert outlet is possible. Outside of the local aggradation at the culvert outlet, no signs of extensive or chronic aggradation were seen. See Section 7.2 for further discussion of aggradation and degradation potential.

2.7.5 Channel Migration

The valley floor of Northeast Dogfish Creek in the project vicinity is approximately 40 to 100 feet wide and valley walls are 20 to 40 feet tall. The active channel is between 9 and 14 feet wide (Table 3). In the existing condition the channel sinuosity is approximately 1.6, with a thalweg distance of 1,488 feet and a valley distance of 950 feet. Channels with slopes over 1 percent have channel migration zones which are approximately equal to the valley width (Rapp, C.F. and Abbe, T.B. 2003).

The reach immediately downstream of the existing crossing is confined against the SR 307 road fill. Aside from this one reach, the rest of the project area is largely unconfined (Section 2.7.2.1). Throughout the entire model domain 100-year flow widths are between 22 and 77 feet wide. In the existing conditions there are no distinct channels on the floodplains; however, modeling results indicate the channel has potential for avulsion or chute cutoff at any of the sharp bends. Deeply undercut banks in the reach immediately downstream of the project crossing indicated the channel is eroding into the bank (Figure 16).

At EX 10+00 in the existing condition the stream makes a hard bend to the north, and there is a large scour pool at this bend (Figure 7). The topography indicates there may have been a historical channel in this location, which rejoined the main channel at EX 8+00.

The channel is predominantly single thread but modeling indicates high flow events spill out onto the floodplain. Due to the unconfined nature of the system there is a risk for channel migration if the channel avulses into a new alignment through the existing floodplain. Although there are indications of an active floodplain, anticipated lateral channel migration is accounted for in the proposed minimum hydraulic width (Section 4.2) and proposed meander bars within the structure (Section 4.3.2) and lateral migration is not expected to be a risk to the structure.

3 Hydrology and Peak Flow Estimates

Chapter 2 of the WSDOT Hydraulics Manual provides guidance for the selection of the most appropriate method of hydrologic analysis (WSDOT 2022a). NE Dogfish Creek is an ungauged basin. Methodologies recommended for ungauged basins are Gauge Basin Transfer with Regional United States Geological Survey (USGS) equations, United States Geological Survey (USGS) Regional Regression Equations, and a Continuous Simulation Hydrologic Model approach. To help put bounds around the uncertainty of modeled hydrology, each method was considered and results were compared.

Basin transfer of gauge data was considered for this project. The watershed of the ungauged stream must be similar in geology, soils, elevation range, vegetation, canopy cover and the size must be within 50 percent of the area of the gauged basin. USGS stream gauges and Kitsap Public Utility District stream gauges were examined, but none that are of the correct size basin or average rainfall have been found. There is a stream gauge on Dogfish Creek that has a watershed area of 5.4 square miles, which is more than double the Project Basin and is outside the allowable limits for the basin transfer methodology.

A gauged basin with a comparable basin size was found in Kitsap County. The Anderson Creek gauge, a stream that drains into Sinclair Inlet, south of Bremerton and west of Port Orchard with data collected just south of SR 16. This gauge has a basin of 1.9 square miles and is within the allowable size limits. However, this gauge is in an area with 52.8 inches annual rainfall which is 130.0 percent of the rainfall that falls in the Project Basin (40.5 inches). The Flood Q Ratio spreadsheet that uses Gauge Basin Transfer weighted with Regional Regression Equations was used to calculate the peak flows even though the annual rainfall is not similar (Table 8). This methodology was not selected, but results are provided for comparison.

Regional Regression Equations require the size of the watershed to be between 0.08 and 2,605 square miles, the mean annual precipitation to be between 33.29 inches and 168 inches, and the percent impervious to be five percent or less (Mastin, et al. 2016). While Crossing 991572 meets the first two criteria, it is over on the allowable impervious area. The NE Dogfish Mainstem basin is 6 percent impervious, the South Tributary basin is 13 percent impervious, and the North Tributary basin is 8 percent impervious (Figure 4). The flow rates from the regional regression equations are shown in Table 8 for a comparison even though these values are outside of the allowable limits of the requirements.

MGSFlood, a Continuous Simulation Hydrologic Model, was used to estimate the design flows for the Project Basin (MGS Engineering, Inc 2018). MGSFlood requires the basin area to be mapped for landcover and soil drainage class. The landcover classifications required for MGSFlood are impervious, forested, pasture, grass, and wetland. The pervious landcover classes (forest, pasture, and grass) are further categorized by their underlying soil units stated as either “till” or “outwash.” To estimate the MGSFlood land cover classes, NLCD land cover classes were grouped into the MGSFlood land cover classes (Table 6). The impervious land cover class was created by subtracting out the impervious percent from each of the other classes (Figure 4). Soils were classified using the USDA Natural Resources Conservation Service’s (NRCS) Web Soil Survey (USDA NRCS 2020). Based on the hydrologic soil groups provided by NRCS (A, B, C, D, A/D, B/D, C/D), each soil unit layer was classified as either till or outwash (Table 7). As assumptions are violated for the Gauge Basin Transfer and Regional Regression methods, MGSFlood is the most appropriate method for estimating peak flows. A comparison of MGSFlood flows to other methods can be seen in Table 8. MGSFlood estimated flows for each subbasin can be seen in Table 9. These are the values that were used in the SRH 2D model.

Table 6. Conversions between NLCD, USDA SSURGO and MGSFlood classifications

NLCD19	MGSFlood
Open Water	Wetland
Perennial Ice/Snow	Wetland
Developed, Open Space	Grass
Developed, Low Intensity	Grass
Developed, Medium Intensity	Grass
Developed High Intensity	Grass
Barren Land (Rock/Sand/Clay)	Grass
Deciduous Forest	Forest
Evergreen Forest	Forest
Mixed Forest	Forest
Dwarf Scrub	Forest
Shrub/Scrub	Forest
Grassland/Herbaceous	Pasture
Sedge/Herbaceous	Pasture
Lichens	Pasture
Moss	Pasture
Pasture/Hay	Pasture
Cultivated Crops	Pasture
Woody Wetlands	Wetland
Emergent Herbaceous Wetlands	Wetland

*Note the Impervious class was created by subtracting area from each of these classes

Table 7. Conversions between USDA Web Soil Service hydrologic soil groups and MGS hydrologic soil groups

USDA	MGSFlood
A	Outwash
B	Outwash
C	Till
B/D	Till
D	Wetland

WSDOT recognizes climate resilience as a component of the integrity of its structures and approaches the design of bridges and buried structures through a risk-based assessment beyond the design criteria. The largest risk to bridges and buried structures will come from increases in flow and/or sea level rise. The goal of fish passage projects is to maintain natural channel processes through the life of the structure and to maintain passibility for all expected life stages and species in a system.

WSDOT evaluates crossings using the mean percent change in 100-year flood flows from the WDFW Future Projections for Climate-Adapted Culvert Design program. All sites consider the 2080 projected percent increase throughout the design of the structure. Appendix G contains the projected increase information for the project site. The design flow for the crossing is 193.5 cubic feet per second (cfs) at the 100-year storm event. The projected increase for the 2080 100-year flow is 61 percent, yielding a 2080 100-year flow of 311.5 cfs (Table 8). The Project Crossing is upstream of the confluence with the North Tributary but the downstream end of the proposed channel regrade carries flow from the North Tributary. Flows from all three basins were modeled in MGSFlood (Table 9).

Table 8. Peak flows for the Project Crossing (NE Dogfish Mainstem and South Tributary basins)

Mean recurrence interval (MRI) (years)	USGS regression equation (Region 3) (cfs)	Gauge Basin Transfer from Dogfish Creek (cfs)	Gauge Basin Transfer from Anderson Creek (cfs)	MGSFlood (cfs)*
2	30	21.3	51.1	**54.5**
10	60.1	42.3	136	**114.2**
25	76.2	52.6	191	**146.9**
50	88.3	59.6	235	**184.0**
100	101	67.9	283	**193.5**
500	133	85.2	404	**199.8**
2080 Projected 100	162.6	137.2	455.6	**311.5**
*Note this table does not include flow from the North Tributary				

Table 9. MGSFlood peak flows for each of the three modeled subbasins

Frequency (year)	NE Dogfish Mainstream (cfs)	North Trib (cfs)	South Trib (cfs)	Combined
2	35.7	6.8	18.8	61.3
10	78.7	12.3	35.5	126.5
25	99.8	17.6	47.1	164.5
50	128.6	23.7	55.4	207.7
100	132.8	24.9	60.7	218.4
500	138.9	33.7	60.9	233.5
2080 Projected 100	212.5	39.8	97.1	349.4

4 Water Crossing Design

This section describes the water crossing design developed for SR 307 MP 1.45 Northeast Dogfish Creek, including channel design, minimum hydraulic opening, and streambed design.

4.1 Channel Design

This section describes the channel design developed for Northeast Dogfish Creek at SR 307 MP 1.45. The design proposes a two-stage channel consisting of a primary bankfull channel with overbank floodplain benches on each side. Both planform and cross-sectional variability will be created with channel complexity features described in Section 4.3.2. The proposed design consists of a constant channel gradient within the restored channel area, with an assumption that localized vertical variability will naturally develop around the forcing features over time.

4.1.1 Channel Planform and Shape

The proposed future conditions channel planform and cross-section shape were informed by the reference reach. The proposed channel geometry is a two-stage channel which targets a design BFW of 12.0 feet (Figure 35). This is slightly smaller than the concurrence average BFW of 12.4 feet (Table 3), but some natural widening is expected to occur as the constructed 2H:1V banks become steeper over time. Channel geometry design has a bankfull depth of 1.4 feet, which matches bank heights observed in the reference reach. The width to depth ratio is 8.5 which falls in the middle of the range (width to depth of 6 to 10) measured in the reference reach.

As this is an unconfined system (Section 2.7.2.1) both a natural conditions and proposed conditions model were built. The natural condition channel planform and shape match the proposed condition, except the floodplain in the natural condition extends all the way out to the assumed historical valley width. The proposed bankfull channel connects to 15H:1V floodplain bench on each side which continues out until it reaches the minimum hydraulic opening (MHO) of 25 feet (Section 4.2). The natural condition is similar, except the floodplains extend out to create a 55-foot valley floor. A 55-foot valley bottom was decided on by estimating where the historical toe of the valley wall was, and by examining hydraulic model results to ensure the 100-year flood extent was not constricted by assumed valley walls. See Section 5.3 for model results.

In the reference reach the two-year flow overtops the bankfull channel by 0.2 feet (Section 5.2). By matching the channel geometry in the refence reach (Figure 36), the proposed design experiences similar overtopping of the main channel and ensures adequate floodplain connectivity. The proposed bankfull depth is 1.4 feet and the modeled 2-year flow is 1.4 to 1.7 feet deep (Section 5.4). The 2-year flow depth slightly overtopping bankfull is consistent throughout the reference reach and proposed channel (see Section 5.2 and Appendix H).

Channel complexity features such as large wood and habitat boulders will create planform variability in both the bankfull channel and thalweg by forcing scour pools and thalweg complexity within the proposed main channel. In later stages of the project, a low-flow channel will be added that connects habitat features together so that the project is not a low-flow barrier. The low-flow channel will be as directed by the engineer in the field.

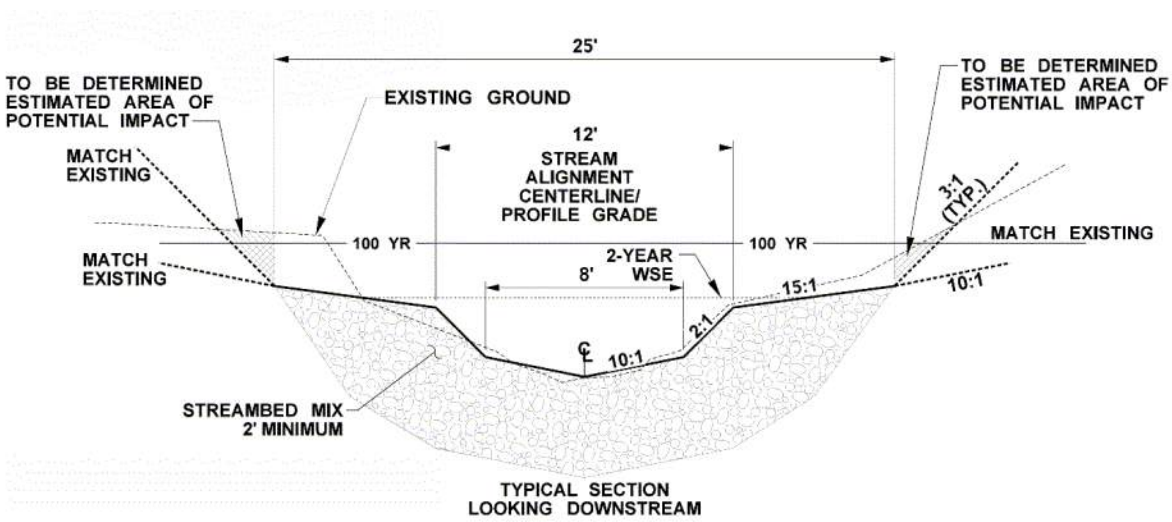

Figure 35. Design cross section

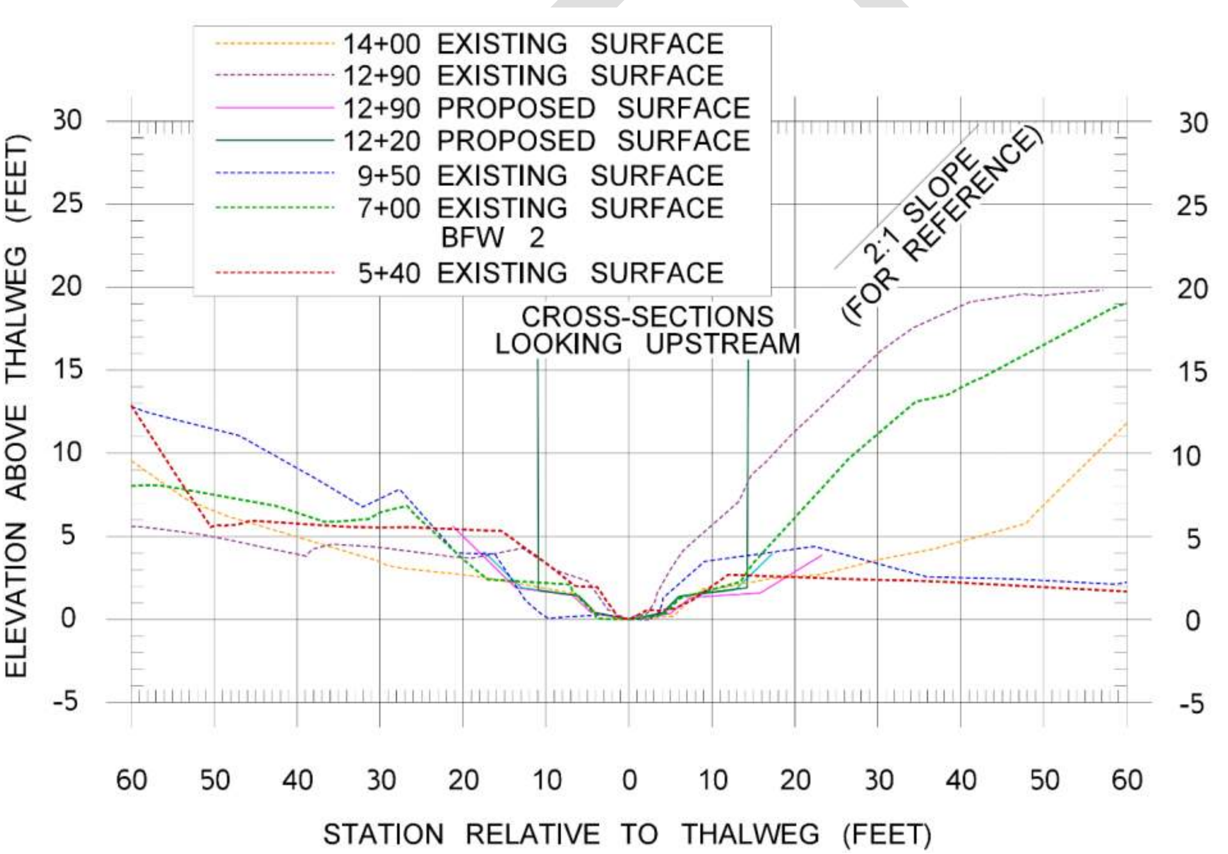

Figure 36. Proposed cross section superimposed with existing survey cross sections

The channel within the vicinity of the project was observed to have meanders. This is common in unconfined systems. Meanders were designed into the proposed planform. In the reference reach and approximately 300 feet upstream of the Project Crossing meanders were mapped and used as the template for meanders in the proposed planform (Figure 37). Copies of the template reach meanders were placed end to end along the approximate centerline of the proposed alignment, then adjusted by hand to create a final single continuous proposed alignment. The meander amplitude in the reference reach and the upstream template reach was 25 feet. This meander amplitude in conjunction with the velocity ratio is the basis for the proposed MHO (Section 4.2.2).

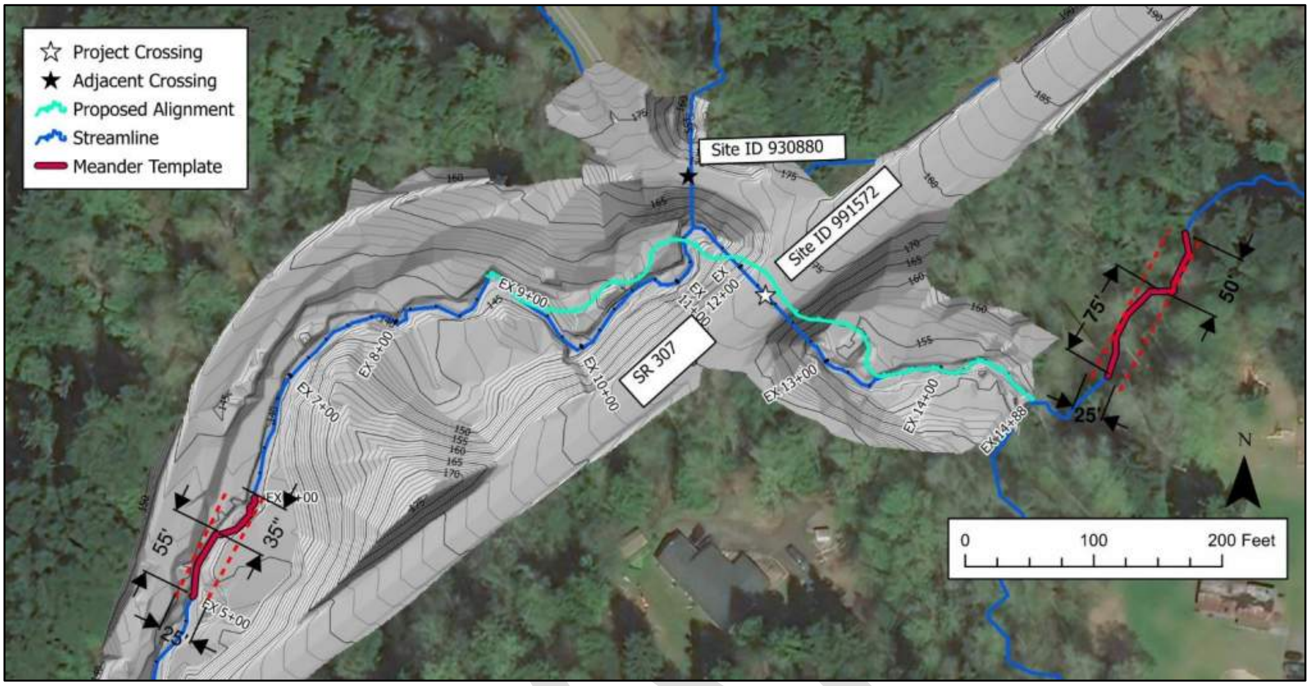

Figure 37. Observed meanders used to design meanders in the proposed planform

Within the crossing there are two meander bars which alternate sides of the stream centerline and create planform variability through the crossing (Section 4.3.2). The proposed meander bar placement approximates the wavelength of meanders observed in the existing channel (Figure 37). Each meander bar is approximately 25 linear feet long at the wall of the crossing structure with a rounded wedge shape that extends toward the crossing centerline (Section 4.3.2.1). The meander bars extend 8.5 feet from the structure walls toward the centerline, such that the edge of the bars extend to the toe of the 12-foot-wide bankfull channel (Figure 43). This alignment was designed to balance channel stability, minimize entrainment along the structure walls, and creating sufficient channel complexity for fish use.

4.1.2 Channel Alignment

The proposed alignment for Northeast Dogfish Creek at the Project Crossings runs southeast to northwest across SR 307 (Figure 38). The total length of the regrade is 374 feet. The proposed channel alignment can be seen in the preliminary design sheets (Appendix D). The proposed alignment pulls the channel away from the private driveway upstream of the crossing and away from the toe of the SR 307 road fill on the downstream side of the crossing. No maintenance records were found for this site, but riprap and pieces of roadway material were found in the channel at the toe of the SR 307 road fill (Section 2.6.2). The proposed alignment moves the channel 10 to 30 feet farther away from existing infrastructure and will reduce necessary maintenance.

The upstream end of the proposed alignment ties in at PR STA 13+03.9 which is 10 feet upstream of where the channel becomes confined against the toe of the valley wall. Moving the channel away from the valley wall will not only protect the private driveway but will also give the channel more room for natural habitat forming processes to occur. Undercut bank habitat will temporarily be reduced but may form again over time. Meanders were built into the proposed alignment with an amplitude and wavelength that approximates meanders seen in the reference reach and upstream of the project site. Measured wavelengths were between 35 and 75 feet, and measured amplitudes were 25 feet (Figure 37).

The proposed regrade ties back to the existing channel approximately 200 feet downstream at PR STA 9+30.0. The tie-in is just upstream of a scour pool which provides high quality habitat and should be preserved. The long regrade downstream of the crossing is largely due to a design goal being to pull the channel away from the valley wall or toe of road fill seen upstream and downstream of the crossing. Discussions during the concurrence meeting indicate moving the channel away from the toe of the SR 307 road fill is a goal for this project (Appendix B).

The existing main channel upstream of the crossing will not be filled and will create disconnected off-channel habitat (see Section 5.4, Figure 68). The abandoned channel will become inundated during the 100-, 500-, and 2080 100-year flow events. The abandoned channel will function similar to a small oxbow lake and provide habitat for amphibians or birds while minimally increasing risk for lateral migration or fish stranding. As flows under the 100-year event do not inundate the abandoned channel, the vast majority of the time it will not be accessible to fish. Channel avulsion into the abandoned channel is unlikely as both the upstream and downstream ends of the abandoned channel will be filled which will discourage flow through the channel.

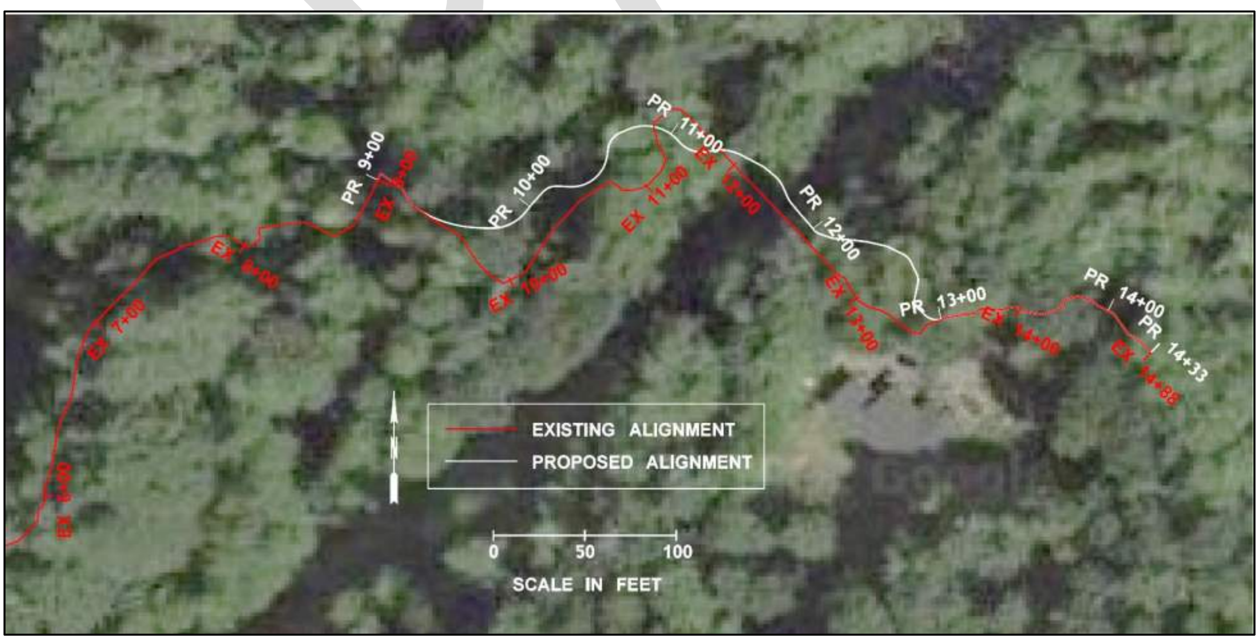

Figure 38. Existing vs Proposed alignments

4.1.3 *Channel Gradient*

The slope of the proposed channel is 2.0 percent throughout the entire length of the regrade. The ground survey shows the slope in the reference reach is 1.7 percent, and the LiDAR shows the slope in a wider vicinity around the Project Crossing is approximately 1.9 percent. The slope ratio, as calculated by the ratio between the proposed channel and the reference reach, is 1.18. The slope ratio as calculated by the ratio between the proposed channel and the survey reach upstream of the crossing (1.6 percent) is 1.25.

The proposed channel matches the slopes found in the existing condition. The proposed channel is expected to remain vertically stable is it is unlikely there will be chronic aggregation or degradation. There is a 3-foot drop in the streambed approximately 1,200 feet downstream of the crossing (Section 2.7.4). If this step migrates back to the crossing it is possible there would be degradation. Scour and deposition on the order of 1 vertical foot near channel complexity features is expected and will help create quality habitat. Refer to Section 7.2 for further discussion on aggradation/degradation.

4.2 Minimum Hydraulic Opening

The minimum hydraulic opening is defined horizontally by the hydraulic width and the total height is determined by vertical clearance and scour elevation. This section describes the minimum hydraulic width and vertical clearance; for discussion on the scour elevation see Section 7. See Figure 39 for an illustration of the minimum hydraulic opening, hydraulic width, freeboard, and maintenance clearance terminology.

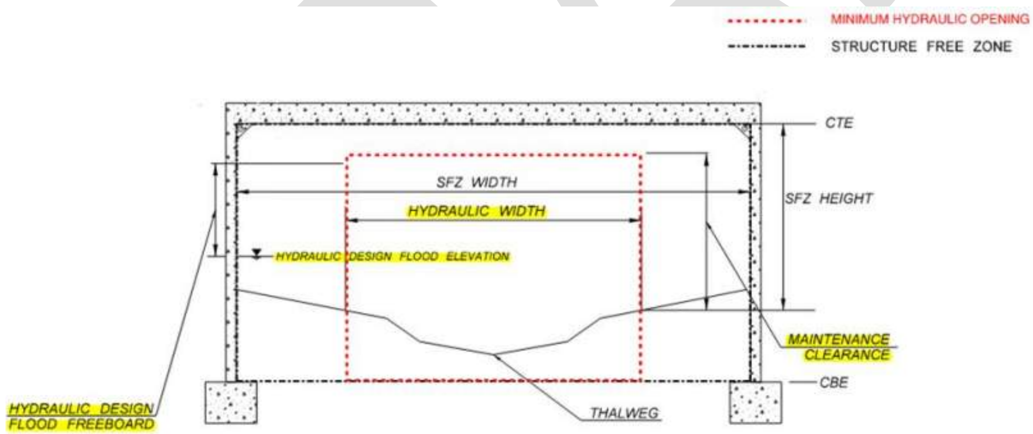

Figure 39. Minimum hydraulic opening illustration

4.2.1 *Design Methodology*

The proposed fish passage design was developed using the WCDG (Barnard, et al. 2013) and the WSDOT *Hydraulics Manual* (WSDOT 2022a). Using the guidance in these two documents, the Unconfined Bridge design method was determined to be the most appropriate at this crossing because the site is classified as unconfined (Section 2.7.2.1). Confinement was quantified using the floodplain utilization ratio (FUR) with a threshold of 3.0. The average FUR throughout the modeled area around the Project Crossing is over 3.0, indicating the site is unconfined.

4.2.2 Hydraulic Width

The starting point for the minimum hydraulic width determination of all WSDOT crossings is Equation 3.2 of the WCDG, rounded up to the nearest whole foot. For this crossing, a minimum hydraulic width of 17 feet was determined to be the minimum starting point. The results of the meander analysis showed that a typical meander amplitude is 25 feet (Section 4.1.1). To allow for the observed geomorphic process of meandering, a modeled hydraulic width of 25 feet was determined to be a more appropriate starting point. As this crossing is unconfined, the hydraulic width is also driven by the velocity ratio. Additional width for lateral migration was not added as accounting for meander amplitude and velocity ratio was considered to add a sufficient factor of safety.

Using the unconfined bridge methodology requires the hydraulic opening to be set using a ratio of velocities between the proposed crossing and the upstream natural condition. The velocity ratio uses the average 100-year main channel velocity through the proposed structure divided by the average 100-year main channel velocity immediately upstream of the structure if the roadway fill were to be removed entirely (i.e., the natural conditions model). The proposed crossing velocity should not be more than 10 percent higher than the existing channel velocity which equates to a velocity ratio no greater than 1.1 (WSDOT 2022a).

Using a minimum hydraulic width of 25 feet, the proposed scenario velocity within the crossing (PR STA 11+70) was 5.5 feet per second and the natural conditions scenario velocity immediately upstream of the crossing (PR STA 11+95) was 5.1 feet per second (Figure 40). This resulted in a velocity ratio of 1.1 which meets the WSDOT requirement (Table 10).

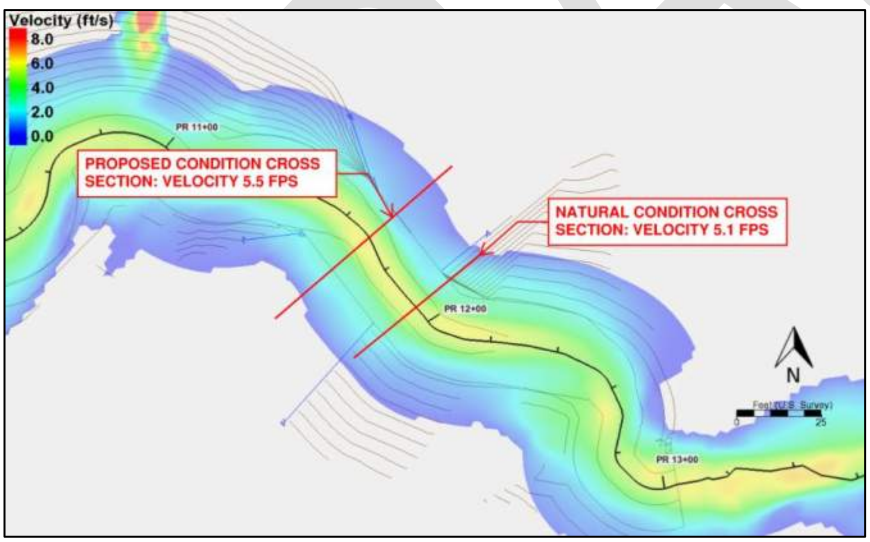

Figure 40. Plan view of cross sections used in velocity ratio

Table 10. 100-year velocity comparison and velocity ratio for 25-foot-wide structure

	Natural conditions upstream cross-section PR STA 11+95	**Proposed conditions through structure cross-section PR STA 11+70**
Average main channel velocity (ft/s)	5.1	5.5
Velocity difference (ft/s)	0.4	
Velocity ratio	**1.1**	

A hydraulic width of 22 feet was modeled as a sensitivity check for the velocity ratio at this site. The 22-foot hydraulic opening width resulted in a velocity ratio of 1.1 which meets the requirement. However, given the possibility of lateral migration upsizing to an opening of 25 feet is recommended as it allows the stream to meander at amplitudes which were measured in undisturbed reaches (Section 4.1.2).

Based on the factors described above, a minimum hydraulic width of 25 feet was determined to be necessary to allow for natural processes to occur under current flow conditions. Table 11 compares the velocities of the 100-year and 2080 projected 100-year events. The 2080 projected 100-year flood is 61 percent higher than the current 100-year flood. This results in approximately a 6 to 15 percent increase in velocity throughout the project area. The velocity increase is approximately 10 percent through the crossing, which is comparable to velocity increases seen in the reference reach. No size increase was determined to be necessary to accommodate climate change. For detailed hydraulic results see Section 5.4.

Table 11. Velocity comparison for 25-foot structure

Location	100-year velocity (ft/s)	Projected 2080 100-year velocity (ft/s)
Reference reach (PR 6+00)	4.8	5.6
Upstream of structure (PR 13+40)	4.9	5.2
Through structure (PR 11+70)	5.5	6.1
Downstream of structure (PR 10+80)	4.8	4.9

4.2.3 Vertical Clearance

The vertical clearance under a structure is made up of two considerations: freeboard and maintenance clearance. Both are discussed below, and results are summarized in Table 12.

The minimum required freeboard at the project location, based on bankfull width, is 2.0 feet above the 100-year water surface elevation (WSE) (Barnard, et al. 2013, WSDOT 2022a). The WSDOT *Hydraulics Manual* and WAC 220-660-190 (4f) require 3 feet of freeboard for all structures greater than 20 feet and on all bridge structures unless otherwise approved by HQ Hydraulics (WSDOT 2022a).

WSDOT is incorporating climate resilience in freeboard, where practicable, and has evaluated freeboard at both the 100-year WSE and the projected 2080 100-year WSE. The WSE is projected to increase by 0.7 to 0.8 feet for the 2080 projected 100-year flow rate. The minimum required freeboard at this site will be applied above the projected 2080 100-year WSE to accommodate climate resilience.

The second vertical clearance consideration is maintenance clearance. WSDOT HQ Hydraulics determines a required maintenance clearance if a height is required to maintain habitat elements, such as boulders or large woody material (LWM). If there are no habitat elements requiring maintenance clearance to maintain, the maintenance clearance is only a recommendation by WSDOT HQ Hydraulics, and the region determines the maintenance clearance required.

The channel complexity features in Section 4.3.2 include boulder habitat features within the structure that may need to be maintained. Therefore, a maintenance clearance of 10 feet to allow for machinery to access and operate under the structure is required. Maintenance clearance is measured from the highest streambed ground elevation within the horizontal limits of the minimum hydraulic width. The maintenance clearance of 10 feet above the highest streambed ground is the required minimum clearance as this clearance exceeds the other requirements described above.

Table 12. Vertical clearance summary

Parameter	Downstream face of structure	Upstream face of structure
Station	PR 11+55	PR 11+91
Thalweg elevation (ft)	148.4	149.1
Highest streambed ground elevation within hydraulic width (ft)	150.3	151.0
100-year WSE (ft)	151.4	152.0
2080 100-year WSE (ft)	152.1	152.8
Required freeboard (ft)	3.0	3.0
Required maintenance clearance (ft)	10.0	10.0
Required minimum low chord, 100-year WSE + freeboard (ft)	154.4	155.0
Required minimum low chord, 2080 100-year WSE + freeboard (ft)	155.1	155.8
Required minimum low chord, highest streambed ground elevation within hydraulic width + maintenance clearance (ft)	160.3	161.0
Required minimum low chord (ft)	**160.3**	**161.0**

4.2.3.1 Past Maintenance Records

WSDOT Area 2 Maintenance was contacted to determine whether there are ongoing maintenance problems at the existing structure because of LWM racking at the inlet or sedimentation. The maintenance representative indicated that there was no record of LWM blockage and/or removal or sediment removal at this crossing.

4.2.3.2 Wood and Sediment Supply

The potential for wood and sediment to be transported into the project reach is considered medium to high. The hydrology modeling indicates peak flows can be nearly 200 cubic feet per second, and that in the future peak flows will increase by an additional 60 percent (Section 3). During the field visits wood was observed accumulated on the inlet of the existing crossing (Figure 8). Much of this appears to be windfall, but there is likely wood which will be transported to the site from upstream reaches. The dominant land cover in the basin is forest (Figure 3) but it is not believed to be heavily impacted by recent logging. Denser housing developments may impact the percent impervious and peak flows in the basin, although the large low slope marsh in the upper watershed will likely provide some attenuation of peak flow increases.

The documented condition of the site in 1999 had a water surface drop at the outlet of the Project Crossing, but this seems to have filled in (Section 2.7.4). The deposition near the outlet is very localized and it is not known if this happened naturally or as part of maintenance activity. Localized and intermittent scour and deposition is expected near placed habitat features. Large woody material will be placed to mimic the flow forcing of wood seen in the existing condition and is not expected to impact aggradation or degradation potential (Section 2.6.4). The recommended 10 feet of vertical clearance is expected to be large enough to allow wood and sediment to be transported through the crossing (Section 4.2.3).

Aggradation or degradation are not expected to become chronic. The one possible exception to this is a four-foot drop in bed elevation noted approximately 300 feet downstream of Site ID 991999. The drop is approximately 1,200 feet downstream of the Project Crossing (STA EX -0+50) and is composed of a mix of sediment, woody debris and live roots (Figure 33). The exposed live tree roots indicate the drop is not completely stable, although the rate of erosion is unknown. See Section 7.2 for more discussion.

4.2.4 Hydraulic Length

A minimum hydraulic width of 25 feet and maximum hydraulic length of 36 feet is recommended. If the hydraulic length is increased beyond 250 feet, the hydraulic width and vertical clearance will need to be reevaluated.

No structure is recommended at this stage, but the crossing was designed assuming the road surface extends out no more than 10 feet from the edge of pavement on either side. This could be accomplished with structures such as large headwalls or with a bridge. The final structure type and length will be determined at a later stage.

4.2.5 Future Corridor Plans

There are currently no long-term plans to improve SR 307 through this corridor.

4.2.6 Structure Type

No structure type has been recommended by WSDOT HQ Hydraulics. The layout and structure type will be determined at later project phases.

4.3 Streambed Design

This section describes the streambed design developed for Northeast Dogfish Creek at SR 307 MP 1.45.

4.3.1 Bed Material

The WCDG (Barnard, et al. 2013) suggest new crossings must be either filled with a material that replicates adjacent channels or be left empty to fill over time. To match the observed streambed sediment, the WSDOT Hydraulics Manual recommends using the stream simulation requirement of a proposed D_{50} within 20 percent of the value in the reference reach (WSDOT 2022a). The WSDOT Hydraulics Manual recommends two approaches for sediment mobility analysis for the proposed sediment mix: the Modified Critical Shear Stress approach for systems with slopes less than 4 percent and the Unit-Discharge Bed Design approach for systems with slopes greater than 4 percent (WSDOT 2022a).

As the proposed crossing has a slope under 4 percent, the Modified Critical Shear Stress methodology was used for assessing stability of all proposed bed material. The goal of the proposed gradation is to match the observed D_{50} within 20 percent. As there seems to be a natural input of sediment from upstream, the design focuses on matching the observed D_{50}. Additional meander bars and habitat boulders will be added to the regraded channel and are expected to help capture naturally transported sediment.

The proposed streambed material mix includes 75 percent Streambed Sediment 9-03.11(1) and 25 percent 6-inch Cobbles 9-03.11(2) (Table 13). The proposed streambed mix has a D_{50} which is within 20 percent of the observed D_{50} (Figure 41). The proposed streambed sediment mix is expected to mobilize at both the 2-year and 100-year events; however, it is expected natural sediment will replace any material transported out of the regraded area.

Table 13. Comparison of observed and proposed streambed material

Sediment size	Observed diameter for design (in)	Streambed sediment (in)	Meander bar tail (in)	Meander bar head
D_{16}	0.3	0.1	0.2	12 to 18-inch habitat boulders
D_{50}	0.8	0.9	1.7	
D_{84}	1.7	2.4	5.0	
D_{95}	3.1	5.0	7.0	
D_{100}	5.1	6.0	8.0	

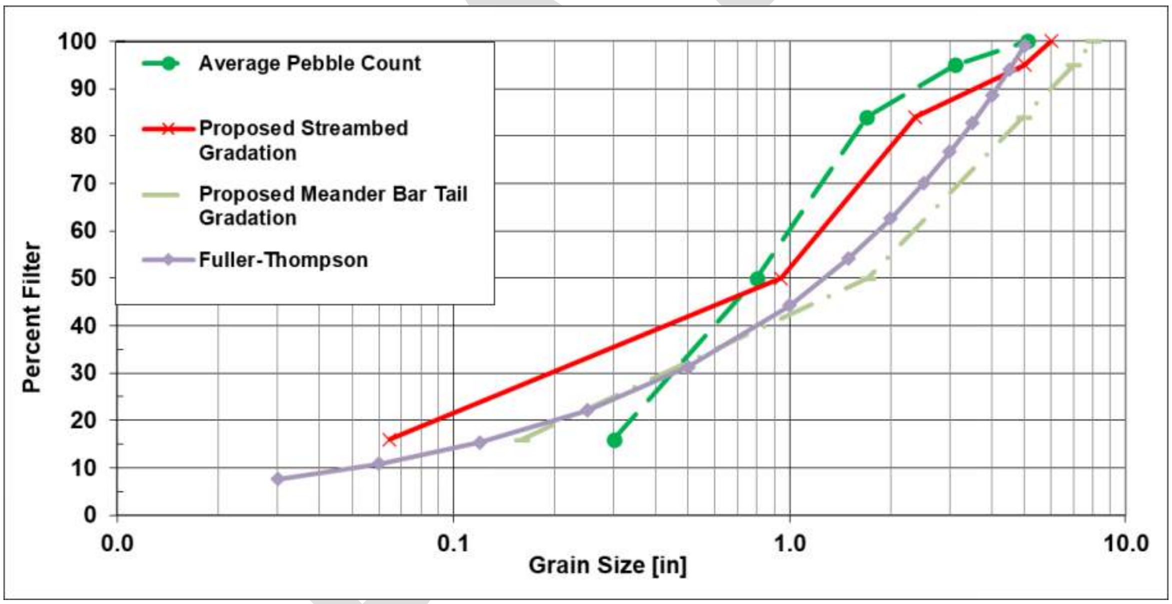

Figure 41. Proposed sediment gradations for the streambed and meander bar tail

Two separate mixes are proposed for the head and tail of the meander bars. Meander bars are designed to maintain the designed channel planform, prevent entrainment along the structure wall and promote habitat complexity and sediment retention. The meander bar head is designed to be immobile up to 100-year event. The meander bar tail mix is designed so that the D_{50} closely matches the existing channel D_{84}, and is expected to remain stable up to the 2-year event. The proposed meander bar tail mix is made up of 50 percent 8-inch Streambed Cobbles mix 9-03.11(2) and 50 percent Streambed Sediment 9-03.11(1). The meander bar head consists of 12-18 inch Habitat Boulders which will be sealed with Streambed Sediment 9-03.11(1). The conceptual layout of the meander bars is shown in Section 4.3.2.1. The meander bar head will be reevaluated for stability at later stages of the design as the final structure size, type, and location are known and the hydraulic model is updated.

It is expected that there will be increased velocity and shear in the vicinity of streambed boulders in contact with the bed and main channel flow. These localized increases should promote deformation (scour holes) of the bed and sorting of particles. Over time, finer sediments will scour and deposit around larger rocks, forming vertical bed complexity. This complexity results in pools for juvenile rearing and adult resting habitat. Local zones of sediment accumulation such as pool tailouts and riffle crests will provide clean, sorted sediment which is suitable for spawning. Riffles will additionally provide juvenile foraging opportunities and increased dissolved oxygen.

The D_{84} of the proposed streambed mix is mobile at the 100-year flow event. Using the Streambed Material Decision tree (WSDOT 2022a) reveals a risk analysis using the Criticality Matrix should be conducted. Risk in this context is defined as the probability of an effect on a constraint, and impact is defined as the level of effect that damage to a constraint would have. Immediately downstream of the project is another fish-bearing crossing (Site ID 930880). Risk of the channel migrating and creating impassible conditions at this crossing is considered low to moderate, and the impact is considered moderate. Matching the observed D_{50} is a higher priority than mitigating this risk and an over-coarsened channel is not recommended. The risk analysis should be reconsidered at the FHD stage when a full stability analysis is conducted.

4.3.2 Channel Complexity

This section describes the channel complexity of the streambed design developed for Northeast Dogfish Creek at SR 307 MP 1.45.

4.3.2.1 Design Concept

The proposed channel design will mimic the reference reach, creating a single-thread two-stage channel which targets the design BFW of 12.0 feet. LWM will be placed at specific locations to develop channel complexity throughout the regraded channel and retain sediment at high flows. The function of the LWM is to enhance habitat in the proposed channel by forming scour pools, providing cover, adding organic material and a source of food, contributing to hydraulic diversity, and encouraging gravel deposition. While a meandering low flow channel will be constructed following direction by the engineer in the field, the proposed LWM design aims to increase channel complexity and fish habitat through natural processes over time. WSDOT has provided guidance and analysis tools for LWM quantities consistent with *A Regional and Geomorphic Reference for Quantities and Volumes of Instream Wood in Unmanaged Forested Basins of Washington State* (Fox and Bolton 2007). There are three metrics representing the LWM quantities observed by Fox and Bolton, density of key pieces, total wood pieces and total wood volume (Table 14). The percentile targets are determined by habitat zone and bankfull width class. The Project Crossing is in the Western Washington habitat zone.

Table 14. LWM Log Metrics (Fox and Bolton 2007)

	No. of key pieces	Total No. of LWM pieces	Total LWM volume (yd^{3})
Design	21	43	150.1
75% Targets	13	43	147.7
50% Targets	7	33	76.1

The key piece density requirement and total number of LWM pieces in the Fox and Bolton (2007) metrics described above were used as the targets for the proposed LWM design. Type 1 and Type 2 logs are key pieces (Figure 42). The minimums required for each metric are based on the total stream grading length of 374 feet. The regrade length for determining quantities includes 55 feet within the crossing structure where LWM will not be placed. See Appendix F for details on the calculations.

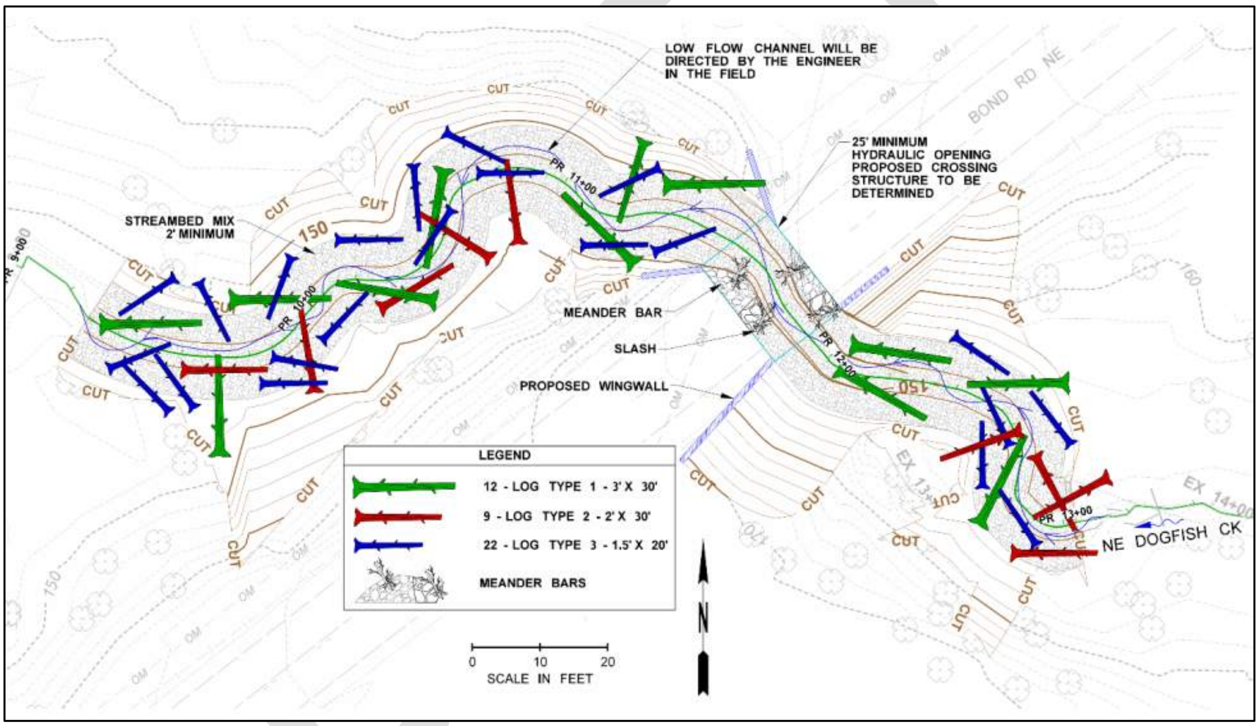

Figure 42. Conceptual layout of habitat complexity

The proposed design meets the 75th percentile targets for number of key pieces and total number of LWM pieces. The proposed LWM layout maximizes the regraded channel area outside the crossing structures and can exceed the 75th percentile target for total LWM volume. The proposed layout adds wood to both the bankfull channel and the floodplain habitat within the channel regrade. Wood stability calculations will be done at the FHD stage of project development and anchoring is anticipated unless stability calculations indicate otherwise.

No large woody material is proposed inside the crossing structure, no mobile wood is placed within 50 feet of the inlet and the wood placement avoids directing flows towards the structure wingwalls. A stability analysis will be done at a later design stage, but it is expected Type 3 logs may mobilize at flows over the 2-year event. Small woody debris within the structure is proposed at the meander bar head. Additional slash is recommended in the tail of the meander bar. Meander bars are proposed within the crossing to increase channel complexity and prevent entrainment (Figure 43). Meander bars within the crossing structure create a channel with increased hydraulic complexity and habitat benefit. Features such as riffles, resting pools, and velocity refuge for rearing juveniles are expected to form as flows are forced by the complexity features. Larger logs with rootwads placed facing the thalweg can ballast smaller logs. This also creates the opportunity for the rootwad to act as an umbrella log and create cover habitat. This cover habitat can replace the undercut bank habitat that will be lost during the channel regrade.

Meander bars force the thalweg to alternate laterally throughout the crossing, preventing entrainment and the formation of shallow, plane bed flow. The spacing of the meander bars allows the channel sinuosity to match the sinuosity measured in the reference reach (Section 4.1.1). Near proposed complexity features the channel is expected to develop local variability such as scour pools and riffles. The channel complexity design provides materials and a preliminary layout designed to encourage formation of the forced pool riffle channel morphology observed in the existing conditions. Preformed pools are not recommended at this stage. Scour pools will provide holding habitat during low and high flows. A low-flow channel directed by the engineer in the field will be built to connect habitat at low flows with the aim of preventing stranding.

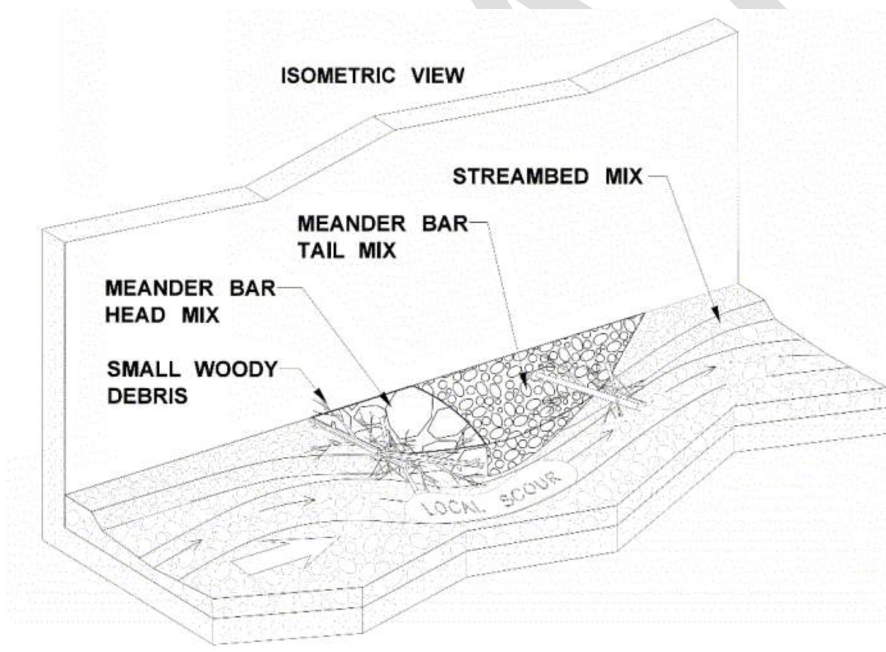

Figure 43. Typical meander bar and habitat boulders

The meander bars will create velocity breaks and a longer flow path as water is forced around each bar during low flow events. The height of the meander bars matches the 10 year flow depth of 2.0 feet. The coarser meander bar head and tail mixes will discourage scouring underneath and force flow back to the center of the crossing. The streambed sediment in the meander bar mixes will fill the gaps between boulders, reducing permeability and increasing stability of meander bars to ensure no risk of fish stranding during low flow conditions. The meander bar design shall be reevaluated at later stages of the design to make sure the latest guidance is implemented.

Although no structure is proposed at this time, the wood layout applies to all anticipated structure types.

4.3.2.2 Stability Analysis

Large wood stability analysis will be completed at final design.

5 Hydraulic Analysis

The hydraulic analysis of the existing and proposed SR 307 Northeast Dogfish Creek crossing was performed using the United States Bureau of Reclamation's (USBR's) SRH-2D Version 3.3.0 computer program, a two-dimensional (2D) hydraulic and sediment transport numerical model (USBR 2017). Pre- and post-processing for this model was completed using SMS Version 13.1.15 (Aquaveo 2021).

Three scenarios were analyzed for determining stream characteristics for Northeast Dogfish Creek with the SRH-2D models: (1) existing conditions with a 4-foot-diameter corrugated round metal culvert, (2) proposed conditions with a 25-foot-wide minimum hydraulic opening, and (3) natural conditions with a 55-foot-wide graded floodplain.

5.1 Model Development

This section describes the development of the model used for the hydraulic analysis and design.

5.1.1 Topographic and Bathymetric Data

The channel geometry data and existing culvert inverts in the model were obtained from the MicroStation and InRoads files supplied by the WSDOT Project Engineer's Office (PEO), which were developed from topographic surveys performed by WSDOT on February 24, 2022. The survey data were supplemented with light detection and ranging (LiDAR) data (DNR 2018). Proposed channel geometry was developed from the proposed grading surface created by PACE using InRoads. All survey and LiDAR information is referenced against the NAVD88 vertical datum.

5.1.2 Model Extent and Computational Mesh

The extents of existing, natural and proposed conditions models are approximately 700 feet upstream and downstream of the existing SR 307 culvert (Figure 44). The upstream and downstream limits of the model are far enough from the regraded channel to not create hydraulic effects at the crossing. Likewise, the boundary conditions (development of which is described in Section 5.1.4) at the upstream and downstream ends of the mesh are at a sufficient distance from the crossing to ensure they will not influence the modeling results at the project crossing. The model extent covers the reference reach, which is located approximately between 440 and 680 feet downstream of the crossing outlet. Survey data ends approximately 200 feet upstream of the existing culvert inlet. LiDAR data was used to extend the model domain upstream of the crossing, as the existing conditions model showed backwatering past the surveyed extent during the 100-year and 500-year events (Figure 57). The extent of the model does not include Site ID 991999. While it is possible there are unrepresented backwater impacts in the existing conditions, no reported model results would be impacted if the backwater was included. For example, any backwatering that covered the reference would require a natural conditions model run to eliminate the backwater. Excluding Site ID 991999 from the model assumes the likely future condition in which Site ID 991999 is replaced and there is no backwater upstream of that crossing.

The hydraulic model's computational mesh must be developed so that important features (channels, roads, etc.) are captured with enough detail that all flows can be modeled accurately. Break lines from the survey data are drawn along these important features to ensure that the mesh represents elevations accurately. The mesh is represented by a network of triangles and quadrilaterals that make up the computational cells (elements) of the model, where the modeling results are computed. Quadrilateral elements are defined in channels, culverts, ditches, and roadway surfaces. Triangular elements are defined in areas where flow may spread in several directions (outlets, floodplains, overbank areas). Nodes comprise the corners of each element. Each node in the mesh has an elevation associated with it, defined from the topographic survey surface. The elevations of nodes around the culvert inlet and outlet were modified to match the surveyed culvert invert elevations to ensure model stability in the existing conditions.

Mesh nodes are spaced along the stream channel at approximately 2-foot intervals so that ten elements span the channel throughout the modeled domain. The mesh developed for the existing conditions hydraulic model has an area of 245,955 square feet and contains 61,224 elements (Figure 44). The mesh developed to model natural conditions covers an area of 243,911 square feet and contains 66,020 elements (Figure 45). Within the regraded extent, the 12-foot-wide bottom channel is spanned by six quadrilateral elements, each 2 feet in width, which is consistent with the existing mesh; floodplains on either side span 21.5 feet and are composed of triangular elements, resulting in a total regraded width of 55 feet following the proposed alignment. The mesh developed for the proposed conditions hydraulic model has six quadrilateral elements spanning the 12-foot-wide bottom channel, with floodplains on either side spanning 6.5 feet composed of triangular elements, similar to the natural conditions model mesh (Figure 46). It was built using 61,081 elements covering the same area as the natural conditions model.

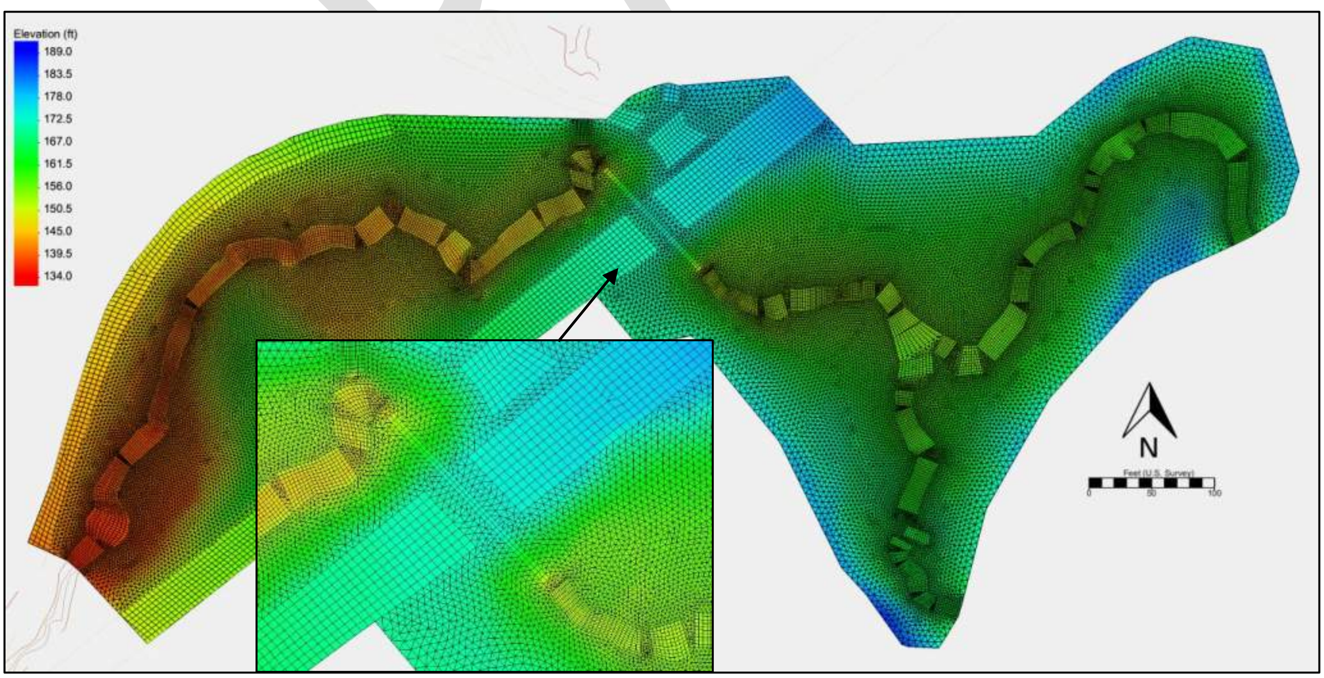

Figure 44. Existing-conditions computational mesh with underlying terrain

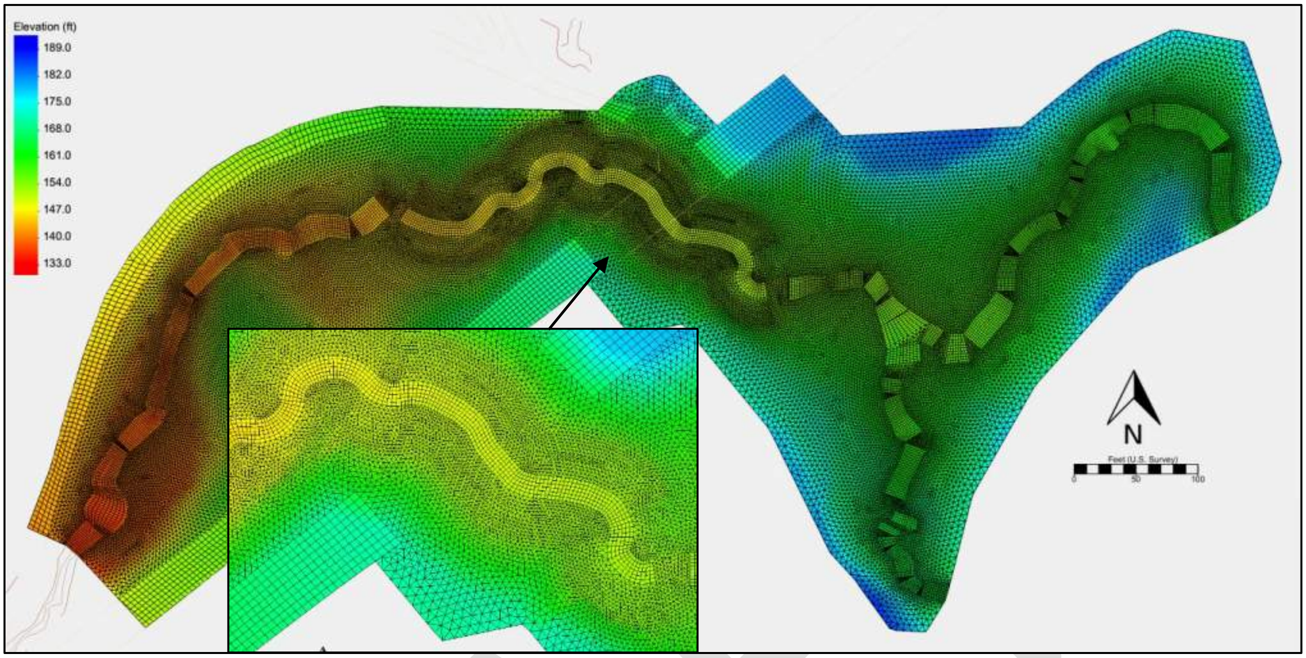

Figure 45. Natural-conditions computational mesh with underlying terrain

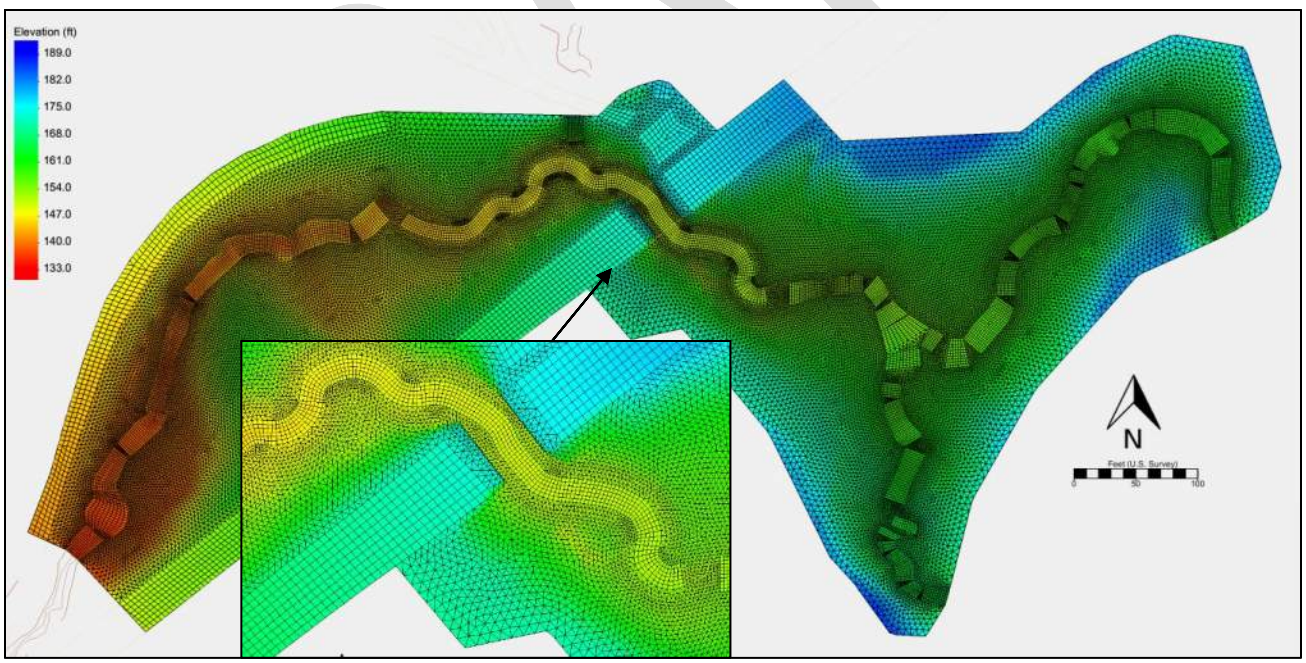

Figure 46. Proposed-conditions computational mesh with underlying terrain

5.1.3 Materials/Roughness

Hydraulic roughness in the SRH-2D model is represented by Manning's n values. These composite values average the roughness within each coverage region. The existing, natural, and proposed model roughness conditions are divided into four categories: main channel, overbank areas, corrugated metal pipe (CMP) culvert, and pavement (Figure 47, Figure 48, and Figure 49). The main channel value was estimated using the U.S. Forest Service's Stream Channel Flow Resistance Coefficient Computation Tool (the spreadsheet tool) (Yochum 2018). The spreadsheet tool combines tabular, semi, quantitative estimates and photographic guidance and provide an overall average roughness value for the main channel. Floodplain values came directly from tabular guidance from the WSDOT Hydraulics Manual. Tabular values with descriptions matching the field observations in Section 0 were chosen. For additional insight into what values were selected to calculate the Manning's values, refer to Appendix E.

An abundance of in-channel LWM and native riparian vegetation was observed at the project site. The proposed channel design includes meander bars and habitat boulders placed inside the crossing and LWM placed outside of the structure throughout the regraded channel. The roughness values for the existing main channel and overbank area reflect the presence of LWM in-channel and native riparian vegetation on the banks. In the natural conditions scenario the overbank area through the crossing is assumed to be consistent with the existing overbank outside of the crossing. In the proposed scenario the meander bars are represented in the sinuous banks, and additional roughness in the overbank area is attributed to shallow flow over the meander bar material.

- **Main channel:** The Manning's n value of 0.054 is assigned to the existing main channel based on the overall average results from the spreadsheet tool. The Hydraulics Manual description of a "Fairly regular section: Dense growth of weeds, depth of flow materially greater than weed height" best fits the existing main channel, resulting in the tabular estimate of 0.05 being used. The degree of irregularity, effect of obstructions, etc., were taken into account by the quasi-quantitative method (Arcement and Schneider 1989) in the spreadsheet tool (Appendix E). A photographic reference for a channel that has similar channel gradient, width, and boulder submergence with the project site provided an estimated roughness value of 0.057 (Yochum 2018). The spreadsheet tool averaged all the estimates into an overall value of 0.054. This composite value was assigned to the main channel in natural and proposed roughness coverages as well, as both are intended to closely mimic naturally occurring conditions in the channel. Note, as the proposed main channel roughness elements are designed to match natural expected conditions and propose roughness values are used in the natural conditions model regrade.
- **Overbank area:** This value of 0.090 was selected from the Hydraulic Manual for "Medium to dense brush: Winter" as this describes the overbank vegetation observed in the field (Figure 11). The meander bars are represented in the mesh, and the tops of the meander bars represents the overbank within the crossing in the proposed condition. A visual reference for a stream with cobbles and occasional boulders was used (Yochum 2018). The overbank value was used in the roughness coverage for existing, natural, and proposed conditions.

- **CMP culvert:** The value of 0.024 was taken from the internal HY-8 tool and assigned to the existing pipe area in the existing roughness coverage. It was not used in the natural or proposed roughness coverages, as the existing culvert is removed in these cases.
- **Pavement:** The Manning's n value of the road, 0.016, is chosen from Open Channel Hydraulics for rough asphalt (Chow 1959). The roughness value for road is assigned in case SR 307 or any nearby roads are overtopped. This value was used for roadways in the existing and proposed roughness coverages, but not in the natural conditions coverage as all roadways are assumed to be removed.

Table 15. Manning's n hydraulic roughness coefficient values used in the SRH-2D model

Material	Manning's n
Pavement	0.016
Existing Main Channel	0.054
Proposed Main Channel	0.054
Overbank Area	0.090
CMP Culvert	0.024

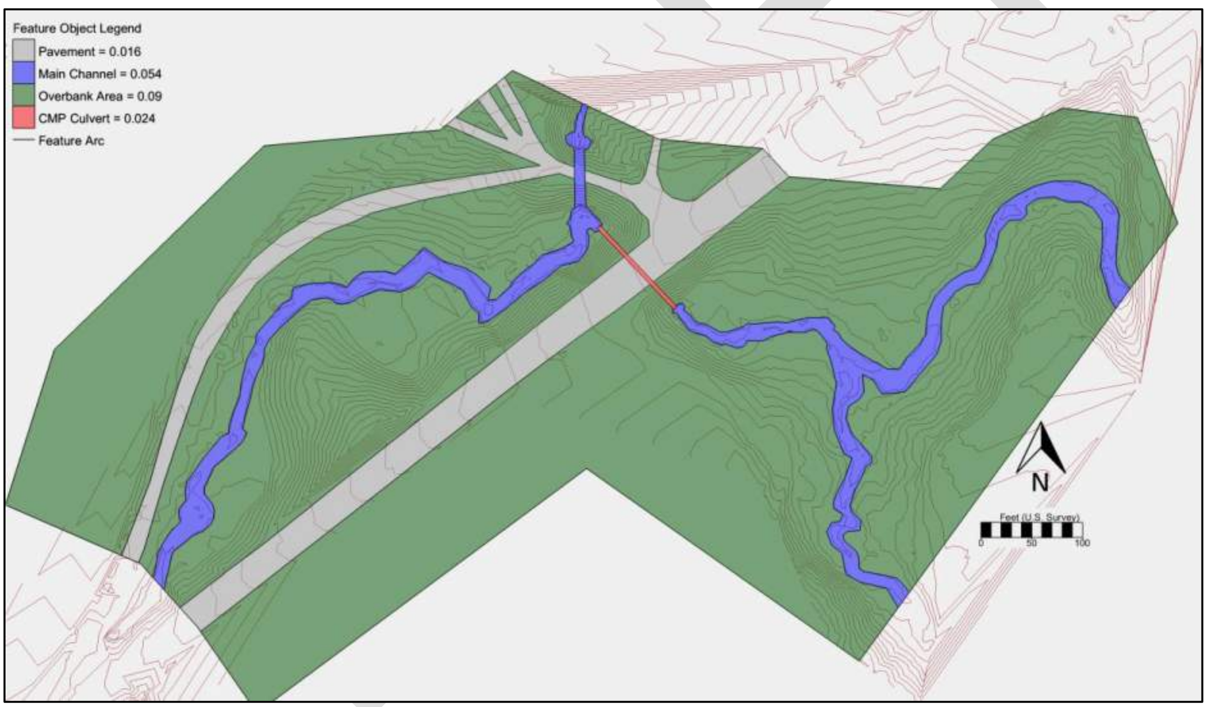

Figure 47. Spatial distribution of existing conditions roughness values in SRH-2D model

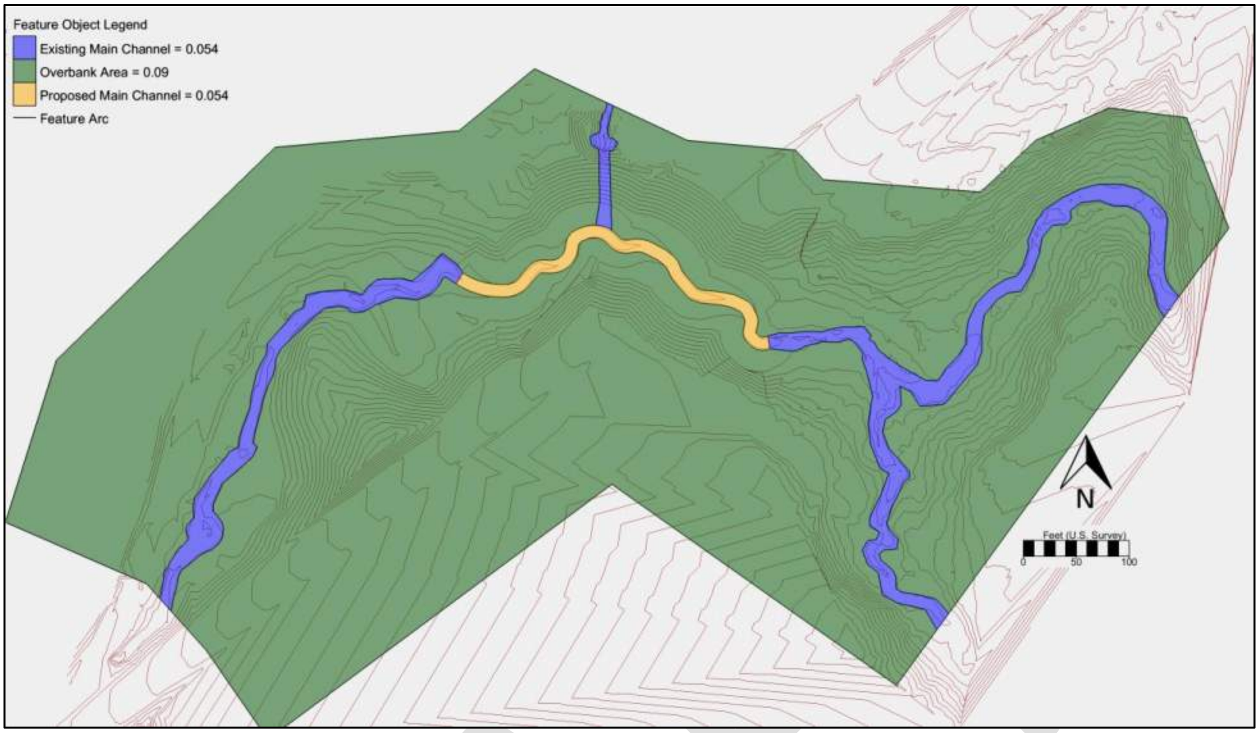

Figure 48. Spatial distribution of natural conditions roughness values in SRH-2D model

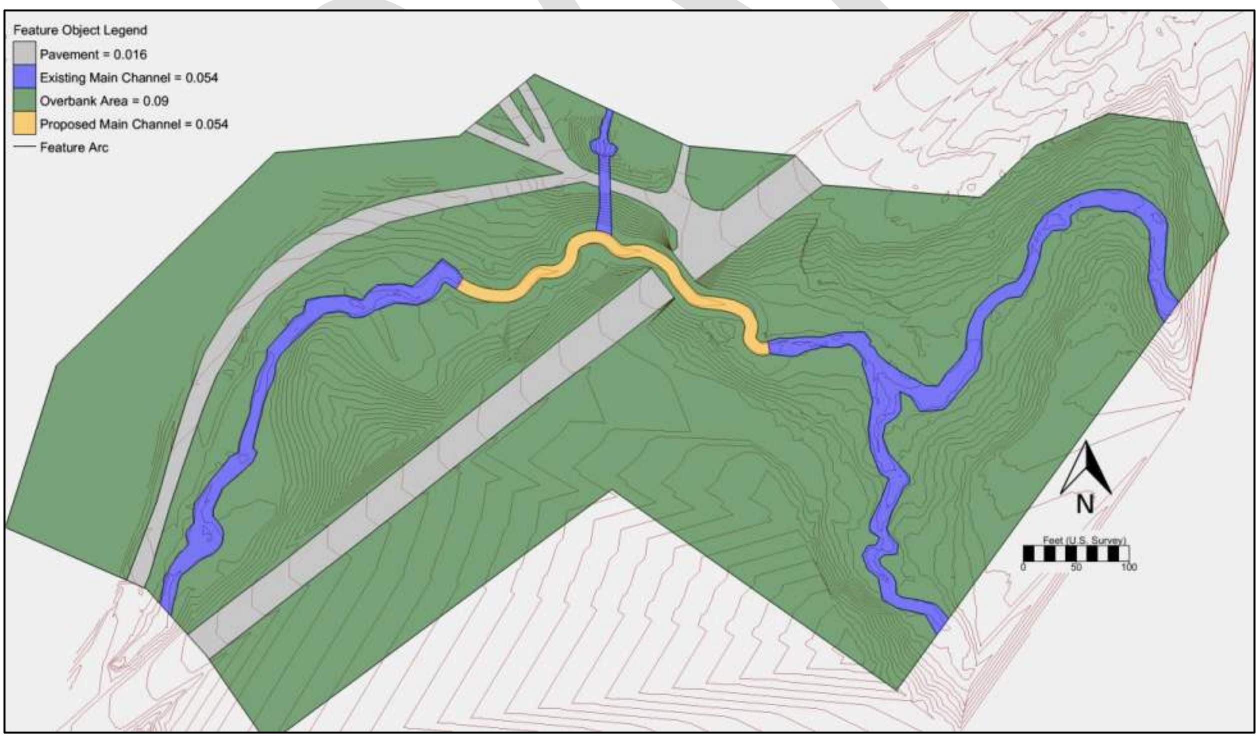

Figure 49. Spatial distribution of proposed conditions roughness values in SRH-2D model

5.1.4 Boundary Conditions

The existing, natural and proposed conditions models were simulated using a steady flow regime. Each model uses the same three basins to estimate inflow hydrology (Section 3). These basins are included in the models as three flow inputs, each represented by a subcritical inflow boundary condition at the edge of the mesh spanning a single flow path. This includes inflow from the “Mainstem” of Northeast Dogfish Creek, which contains the majority of flow, inflow from the “South Tributary,” which enters the model from the south and joins the mainstem approximately 200 feet upstream of the crossing inlet, and inflow from the “North Tributary,” which enters the model from the north and joins the mainstem approximately 20 feet downstream of the crossing outlet. The placement and flow values of each inflow boundary condition were kept constant across the existing, natural, and proposed condition models. The flows used are summarized in the hydrologic analyses in Section 3. For this model separate Boundary Condition coverages were used for each flow regime and a new simulation was run for each flow.

The exit boundary condition is a subcritical outflow using a constant WSE calculated through normal depth equation. The model inputs are a slope, roughness and discharge. A slope input of 0.017 was used based on surveyed bathymetry data. A Manning’s n value of 0.054 was used based on the roughness of the Main Channel in Figure 47. The flow is determined from the sum of all input flows for each storm event: 2-year 61.3 cfs, 100-year 218.4 cfs, 500-year 233.5 cfs, and 2080 projected 100-year 351.6 cfs. These values were selected so that the entirety of the flow would be captured at the exit. See Figure 51 for the 100-year exit boundary condition calculation. These boundary conditions were used in the existing, natural, and proposed conditions models.

The existing 4-foot-diameter culvert is simulated using HY-8 extension (Aquaveo 2019) through boundary conditions in SMS (Aquaveo 2021) in the existing conditions model. The input parameters can be found in Figure 50. This information is from the survey data delivered by the WSDOT survey team in February 2021. The Manning’s n value of the crossing is provided by HY-8 based on the corrugated metal pipe input.

All the models contain the three inflow boundary conditions and outflow boundary conditions described in the preceding paragraphs. The existing condition model additionally contains the HY-8 culvert boundary condition. The locations of the boundary conditions in the existing, natural, and proposed conditions models are shown in Figure 52, Figure 53, and Figure 54 respectively.

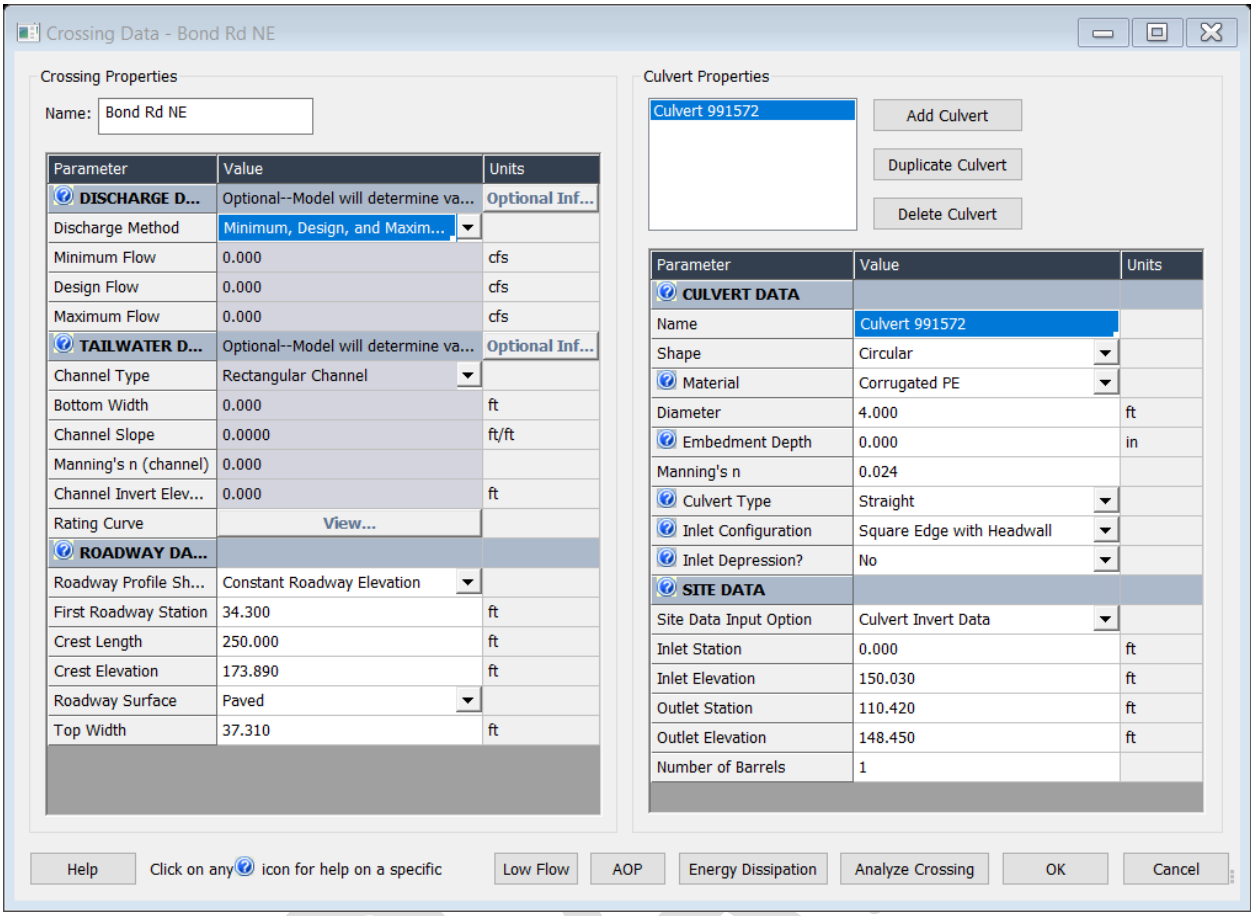

Figure 50. HY-8 culvert parameters

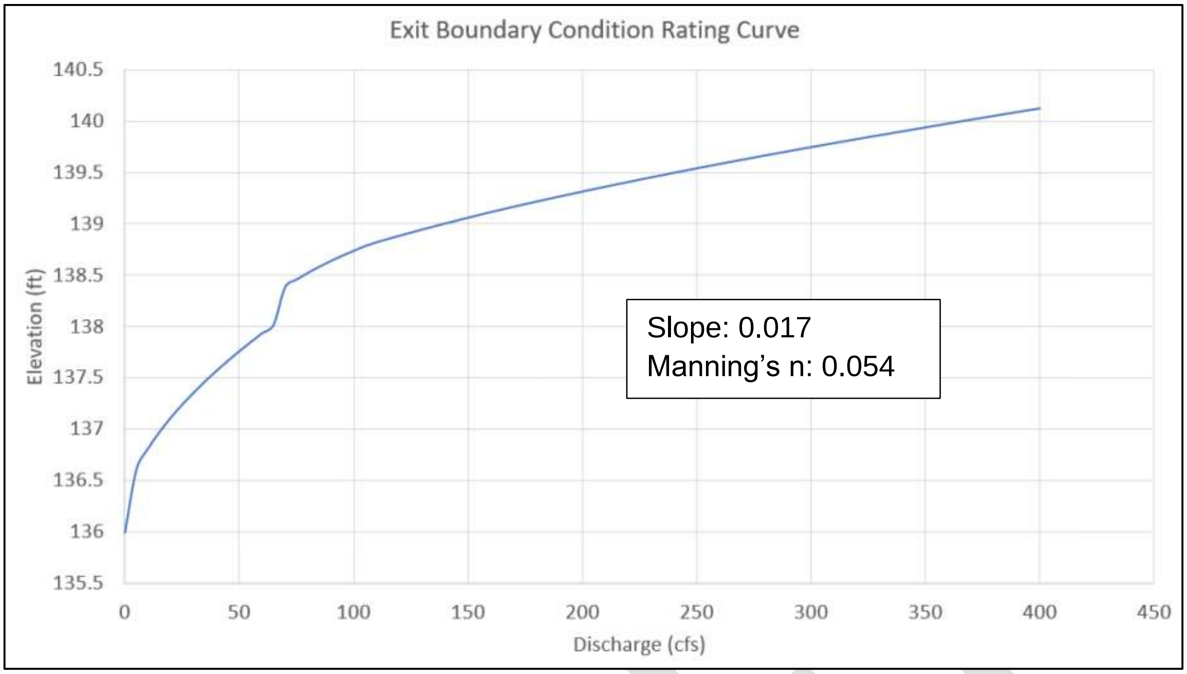

Figure 51. Downstream outflow boundary condition normal depth calculation

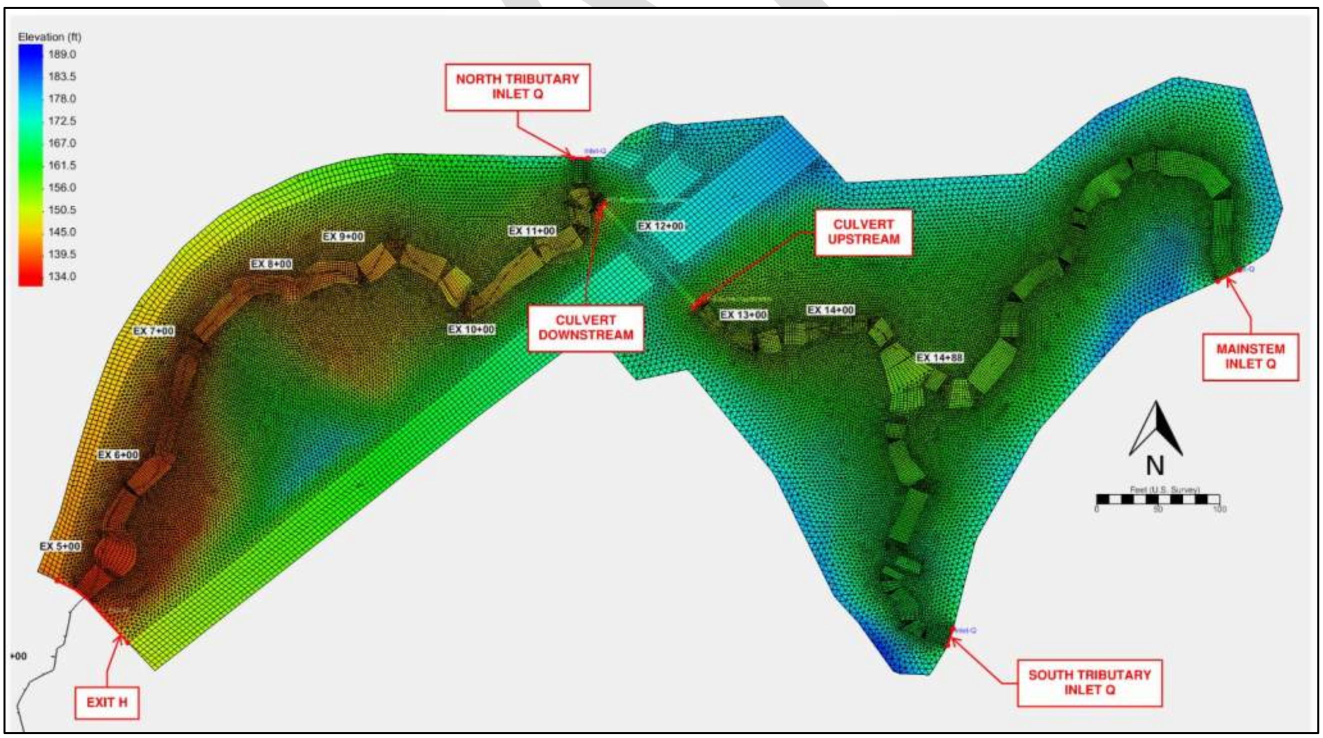

Figure 52. Existing-conditions boundary conditions

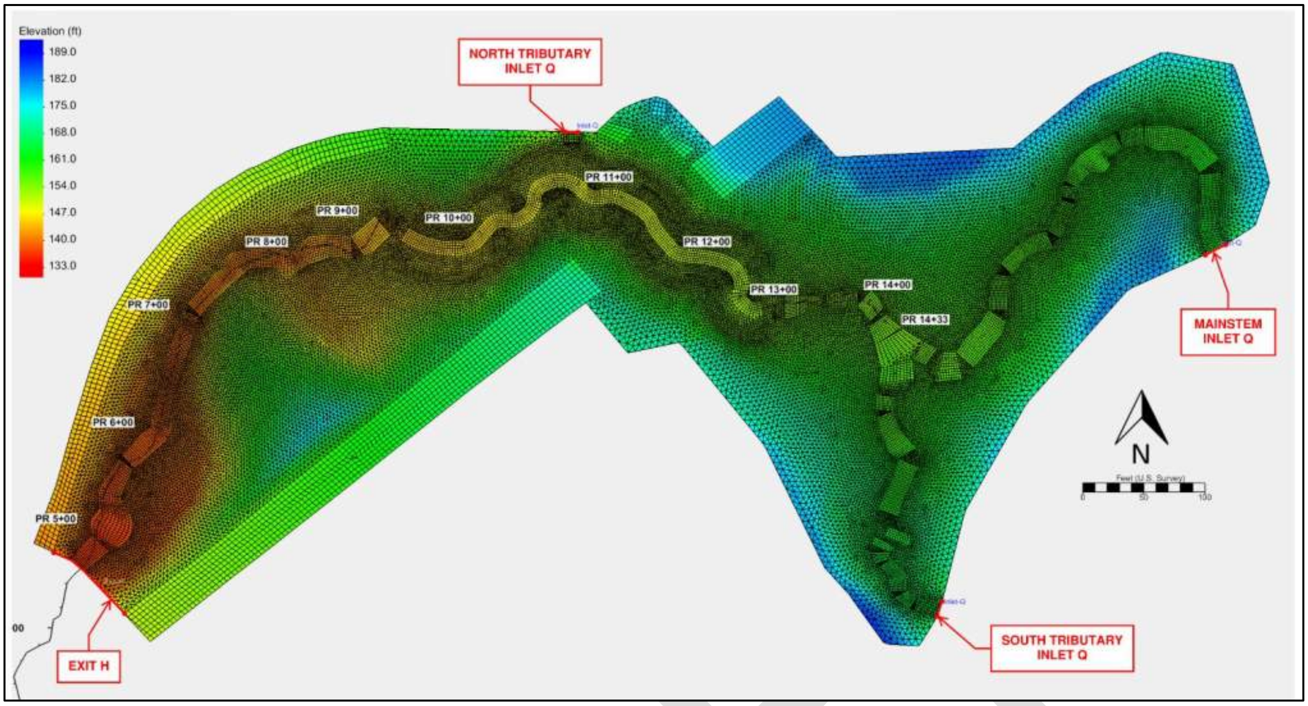

Figure 53. Natural-conditions boundary conditions

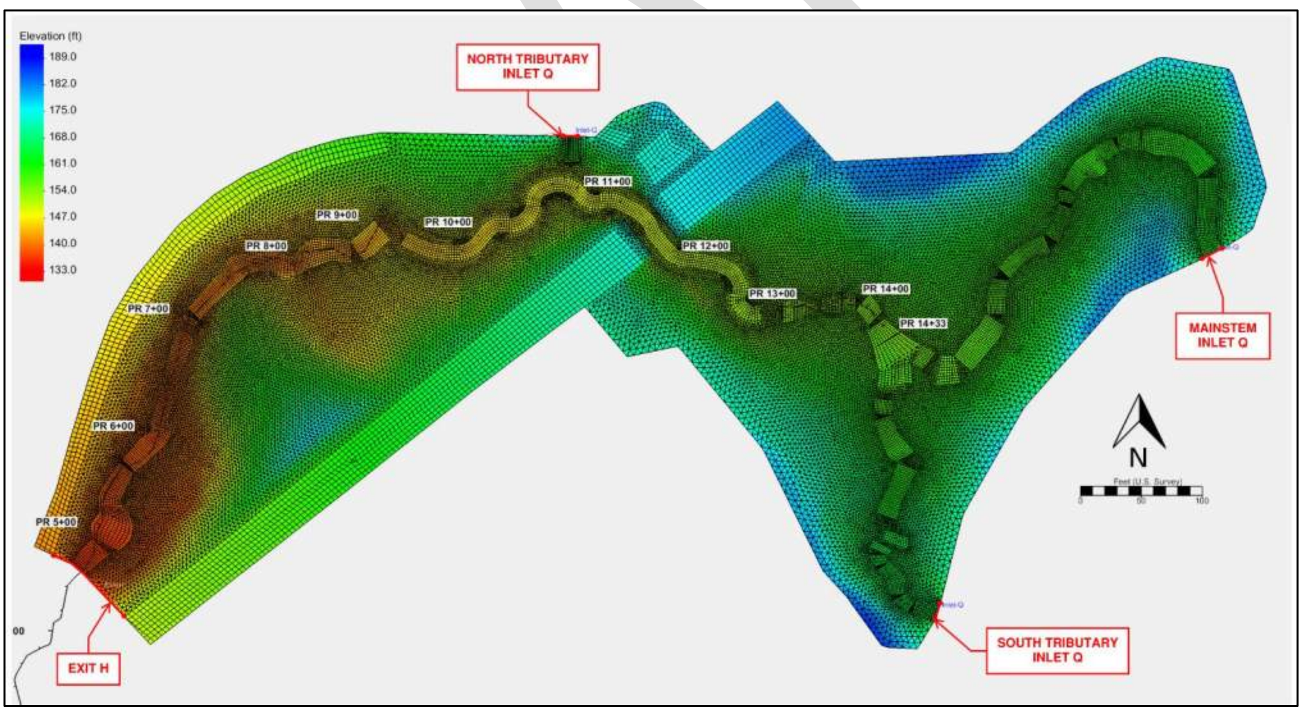

Figure 54. Proposed-conditions boundary conditions

5.1.5 Model Run Controls

All of the simulations reported in this document use a dry initial condition, a time step of 0.5 seconds, and a start time of 0.0 hours. Each of the flow scenarios within the natural and proposed conditions models have an end time of 1.0 hours, during which the flows comfortably converge and reach steady state. The existing 2-year storm event is also given a 1.0-hour runtime, during which the flows converge and stabilize. Heavy backwatering occurs during 100-year and 500-year storm events in the existing conditions model due to the undersized culvert restricting flow; because of this, an 8.0-hour runtime is required for the 100-year event, and a 10.0-hour runtime for the 500-year event. For the existing 100-year and 500-year storm scenarios, the flows converge shortly past the 7.0-hour mark and 8.5-hour mark, respectively, and maintain steady state during the remaining time in the simulation. See Appendix I for monitor line plots.

5.1.6 Model Assumptions and Limitations

The complexity of small-scale hydraulics which form around LWM and meander bars are not accurately simulated in the 2-dimensional (2D) model. These micro-scale hydraulics are beyond the scope of the PHD investigation and require different software packages with greater computational demand and level of effort to develop. Meander bars are accounted for in the mesh and the sinuous channel through the crossing. It is assumed the additional hydraulic complexity and roughness provided by the meander bars within the structure matches the hydraulic complexity and roughness provided by LWM outside of the structure.

The proposed alignment for this project differs from the existing alignment. See Figure 55 for a comparison of stationing.

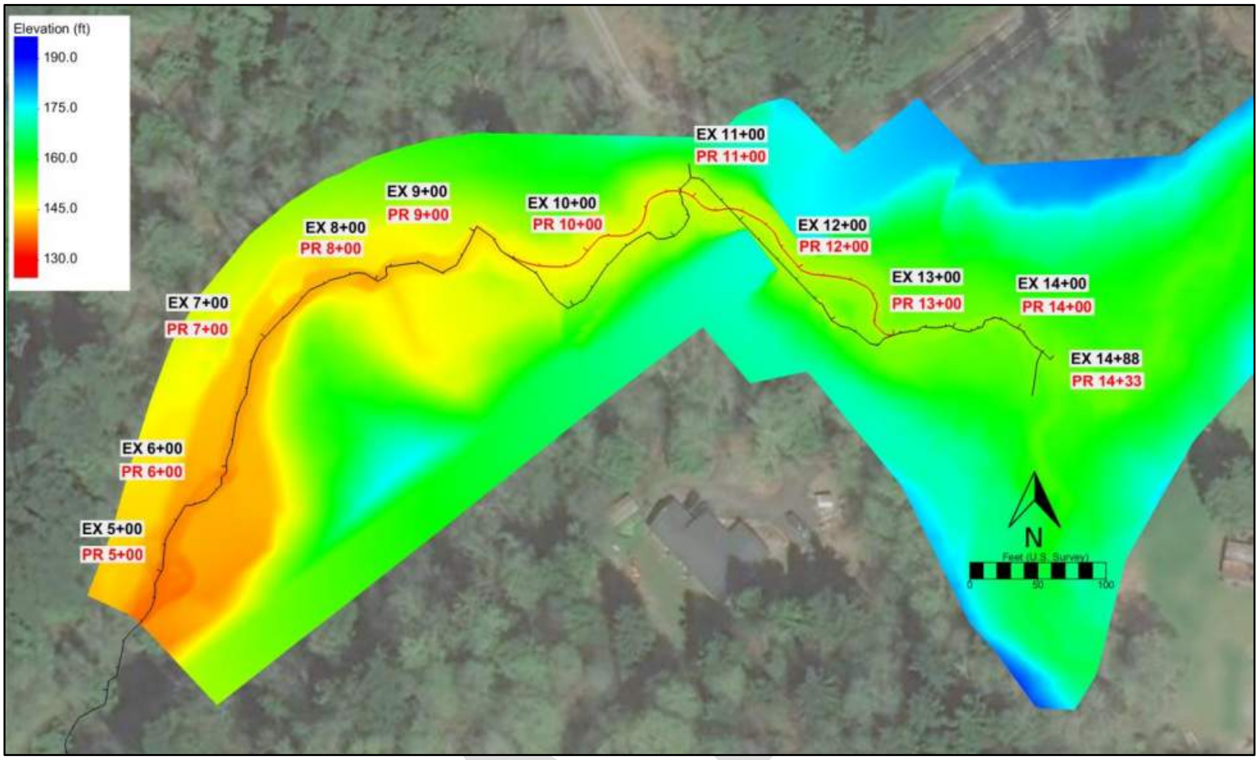

Figure 55. Existing (black) versus proposed (red) alignment

5.2 Existing Conditions

The 2-, 100-, and 500-year peak flow events were simulated in the existing conditions model. Inundation and flow characteristics were extracted from the model at selected cross-section locations shown in Figure 56, with the results shown in Table 16. Cross-sections were drawn upstream and downstream of the existing crossing to observe the hydraulic impact of SR 307 construction in the existing conditions. Additional cross-sections were taken in the reach upstream of the culvert inlet and throughout the reference reach downstream of the outlet. Appendix H contains additional cross-sectional plots as well as plan view figures of hydraulic model results. WSE profiles along the thalweg can be seen in the stream profile below in Figure 57. Heavy backwatering extending approximately 650 feet upstream of the culvert inlet can be observed at the 100-year and 500-year flow events. This backwatering is due to the undersized existing culvert, and is roughly 15 feet deep at the culvert inlet. During the 500-year event, high shear stress and velocity values of 25 pounds per square foot and 16 feet per second, respectively, were observed at the north tributary basin inlet, which intercepts the channel directly downstream of the culvert outlet. This is most likely due to the steep slope of the incoming channel, and does not affect main channel results.

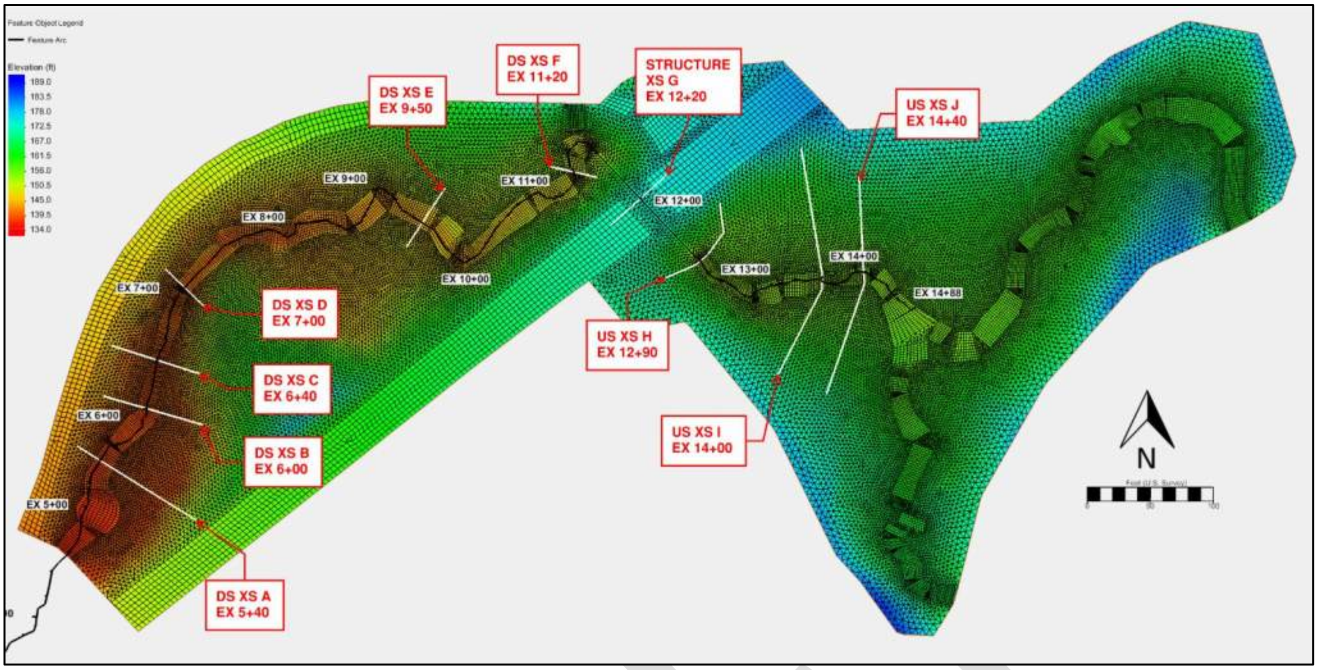

Figure 56. Locations of cross sections used for results reporting

Figure 58 shows the modeled flows at a cross-section upstream of the culvert; it should be noted that this cross-section falls within the backwatered extent and is not expected to represent the natural condition of the channel. Natural conditions model results in comparable locations along the proposed alignment can be found in Section 5.3. Figure 60 shows WSE results within the reference reach. Northeast Dogfish Creek is an unconfined channel, with wide floodplains throughout the reference reach and 2-year flow mostly contained within the top of banks. The existing conditions 100-year velocity map and overbank velocity values can be found in Figure 60 and Table 17, respectively.

Hydraulic model results for the reference reach in the existing conditions analysis done for Site ID 991999 (the next crossing downstream) were compared to the exiting conditions analysis done for Site ID 991572. Cross sections were taken at slightly different locations between the two sites so there is no direct comparison. Model values are reported for a cross section at EX 6+40 for 991572 and EX 6+51 for 991999. At this location the WSE difference is under 0.2 feet, and the velocity difference is under 0.4 feet per second.

Table 16. Average main channel hydraulic results for existing conditions

Hydraulic parameter	Cross section	2-year	100-year	500-year
Average WSE (ft)	DS EX 5+40 (A)	139.6	140.6	140.7
	DS EX 6+00 (B)	140.4	141.4	141.4
	DS EX 6+40 (C)	141.2	142.2	142.3
	DS EX 7+00 (D)	142.4	143.4	143.5
	DS EX 9+50 (E)	145.7	147.0	147.2
	DS EX 11+20 (F)	149.6	151.1	151.2
	Structure (G)	NA	NA	NA
	US EX 12+90 (H)	153.5	164.9	165.7
	US EX 14+00 (I)	154.0	164.9	165.8
	US EX 14+40 (J)	154.4	164.9	165.8
Max depth (ft)	DS EX 5+40 (A)	1.7	2.7	2.8
	DS EX 6+00 (B)	1.7	2.6	2.7
	DS EX 6+40 (C)	1.9	3.0	3.0
	DS EX 7+00 (D)	2.0	3.0	3.1
	DS EX 9+50 (E)	1.4	2.7	2.9
	DS EX 11+20 (F)	1.4	2.9	3.0
	Structure (G)	NA	NA	NA
	US EX 12+90 (H)	2.6	13.9	14.8
	US EX 14+00 (I)	2.1	12.9	13.8
	US EX 14+40 (J)	2.1	12.6	13.4
Average velocity (ft/s)	DS EX 5+40 (A)	3.7	4.8	4.9
	DS EX 6+00 (B)	2.9	4.8	4.9
	DS EX 6+40 (C)	3.2	5.3	5.5
	DS EX 7+00 (D)	3.0	5.7	5.9
	DS EX 9+50 (E)	3.6	5.4	4.9
	DS EX 11+20 (F)	4.6	6.0	6.1
	Structure (G)	NA	NA	NA
	US EX 12+90 (H)	2.8	1.3	1.3
	US EX 14+00 (I)	2.7	0.2	0.2
	US EX 14+40 (J)	3.3	0.2	0.2
Average shear (lb/SF)	DS EX 5+40 (A)	1.3	1.7	1.7
	DS EX 6+00 (B)	1.0	1.7	1.8
	DS EX 6+40 (C)	0.8	1.8	1.9
	DS EX 7+00 (D)	0.7	2.1	2.2
	DS EX 9+50 (E)	1.3	2.0	1.8
	DS EX 11+20 (F)	2.2	2.7	2.8
	Structure (G)	NA	NA	NA
	US EX 12+90 (H)	0.5	0.1	0.1
	US EX 14+00 (I)	0.6	0.0	0.0
	US EX 14+40 (J)	0.9	0.0	0.0

Main channel extents were approximated by surveyed top of bank lines.

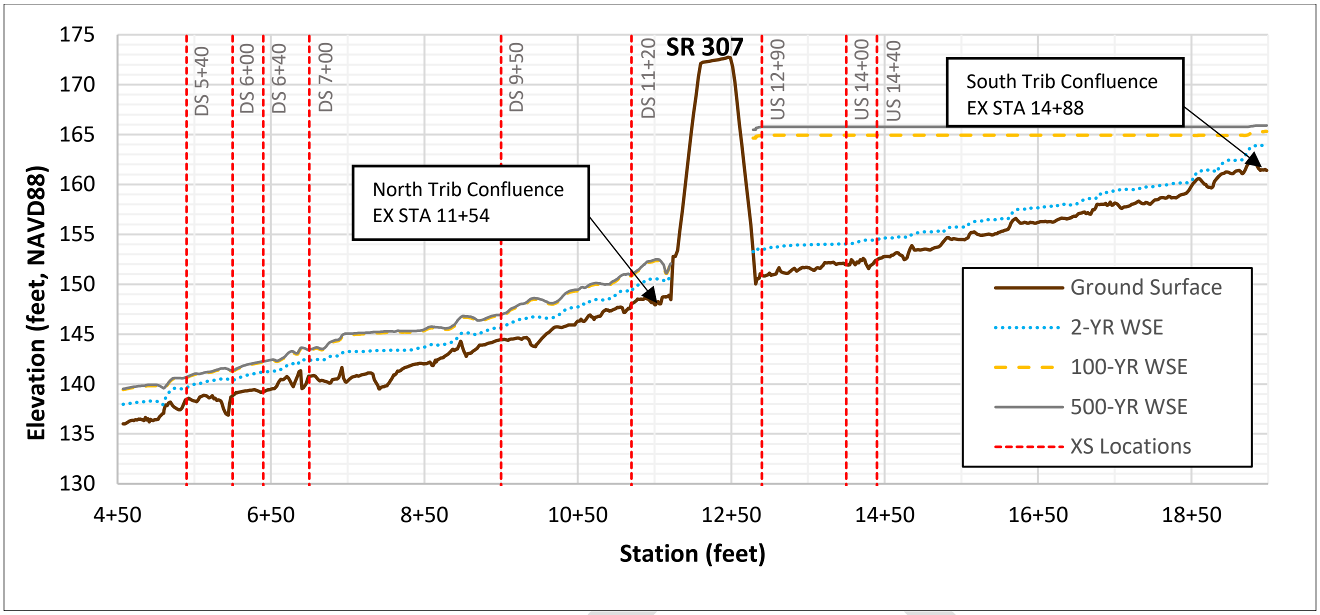

Figure 57. Existing-conditions water surface profiles

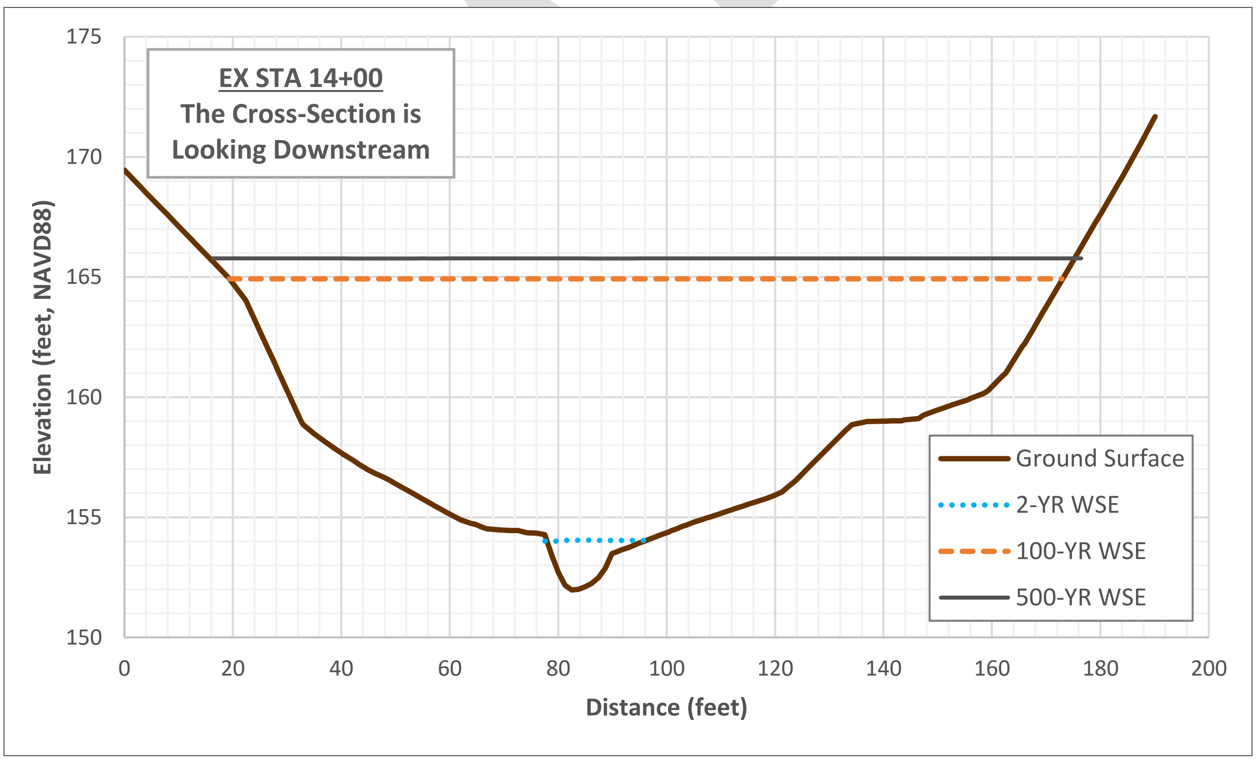

Figure 58. Typical upstream existing channel cross section (EX STA 14+00)

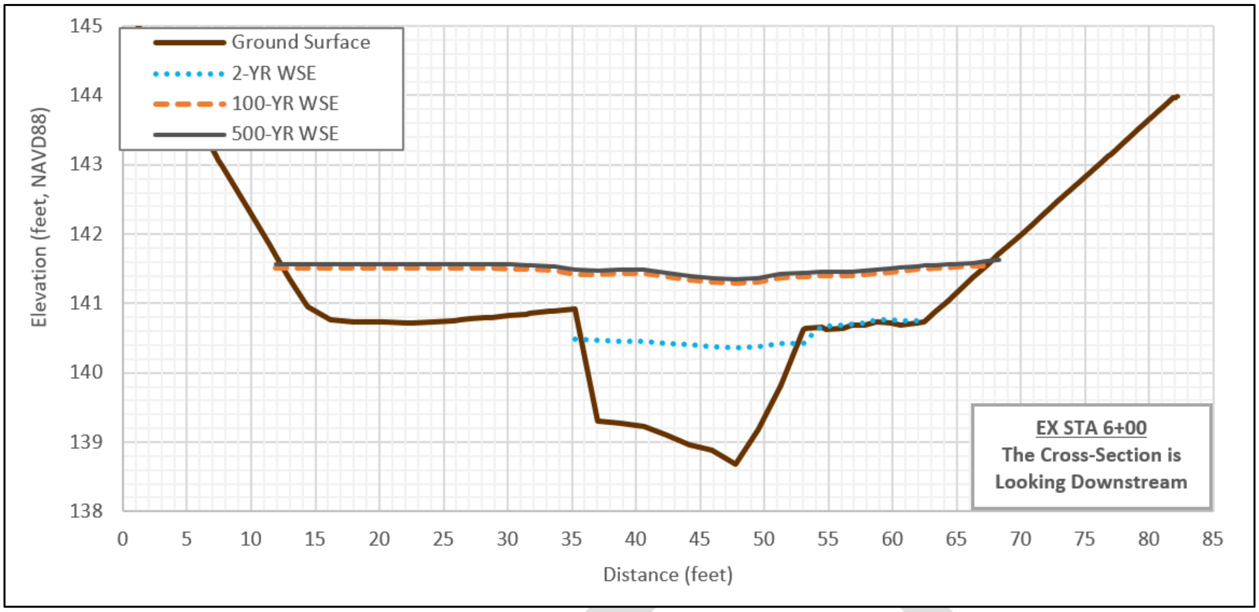

Figure 59. Typical reference reach cross section (EX STA 6+00)

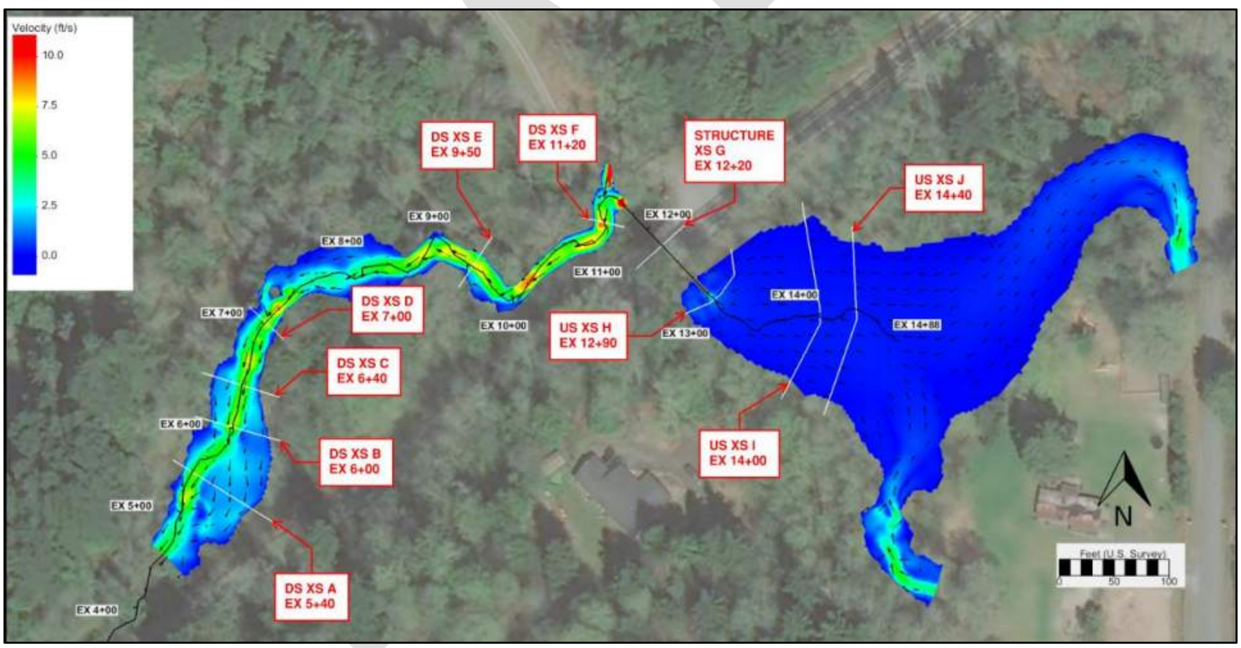

Figure 60. Existing-conditions 100-year velocity map with cross-section locations

Table 17. Existing-conditions average channel and floodplains velocities

Cross-section location	Q100 average velocities tributary scenario (ft/s)		
	LOB[a]	Main channel	ROB[a]
DS EX 5+40 (A)	1.6	4.8	2.2
DS EX 6+00 (B)	1.2	4.8	2.5
DS EX 6+40 (C)	1.9	5.3	1.9
DS EX 7+00 (D)	2.8	5.7	0.9
DS EX 9+50 (E)	1.2	5.4	NA
DS EX 11+20 (F)	NA	6.0	3.6
Structure (G)	NA	NA	NA
US EX 12+90 (H)	0.8	1.3	0.3
US EX 14+00 (I)	0.2	0.2	0.1
US EX 14+40 (J)	0.2	0.2	0.1

Right overbank (ROB)/left overbank (LOB) locations were approximated by surveyed top of bank lines.

5.3 Natural Conditions

A 55-foot-wide graded channel and floodplain following the proposed alignment was utilized to model natural conditions in the absence of roadway fill and existing structures. Outside of the 55-foot-wide regraded area, 2:1 grading extends outward until the tie-in location is reached; this results in a minimum of 30 feet of grading on either side of the channel through the crossing, which effectively removes the hydraulic effect of the existing roadway. See Section 4.1.1 for details on the natural conditions design. The 2-, 100-, 2080 100-, and 500-year peak flow events were simulated in the natural conditions model. Inundation extents and results extracted at selected cross-section locations are discussed in the following paragraphs. Appendix H contains additional cross-sectional plots as well as plan view figures of hydraulic modeling results.

The natural conditions model results extracted from the selected cross-sections A through J are shown in Table 18. The locations of the cross-sections along the proposed alignment are shown in Figure 61. Cross-sections in the natural model are located at the same place as they are in the existing model but shifted slightly to be centered on the proposed alignment and extend past the width of flow for each storm event. Stations are identical in the downstream reference reach but vary upstream of STA 9+30 due to the difference between the existing and proposed alignments. Model results show that the 2-, 100-, and 500-year flow events are adequately contained within the 55-foot-wide channel; flow only exceeds the 55-foot width in the 2080 projected 100-year scenario and is contained within the 2:1 side grading. The 100-year flow width through the crossing location is 51.8 feet, with a 2-year width of 17.4 feet. The 2-year flow inundation extent within the reference reach is approximately at the survey top of bank line. This model result validates the peak flow estimates. The backwatering shown at the culvert inlet in the existing conditions scenario is eliminated in the natural scenario (Figure 62).

With the hydraulic effects of the roadway fill and existing structure removed, the natural conditions model was used to confirm the unconfined nature of the channel. Cross-sections were taken at uniform 50-foot spacing outside of the regraded channel, resulting in an average FUR of 3.5. For a full FUR analysis, see Section 2.7.2.1.

As illustrated in Figure 63, the 2-year flow is nearly contained within the main channel. This is a consistent trend at all cross-sections taken throughout the modeled domain and is similar to modeling results for existing conditions, where 2-year flow slightly exceeds the main channel (Figure 58). Main channel average velocity hovers consistently around 3.7 to 5.6 feet per second throughout the modeled domain during the 100-year flow event, and 4.1 to 6.6 feet per second during the 2080 projected 100-year flow event. The floodplain velocities are relatively slow, ranging from 0.3 to 2.5 feet per second during the 100-year event and 0.7 to 3.8 feet per second during the 2080 projected 100-year event. The maximum 100-year velocity of 12.7 feet per second occurs at the north tributary basin inlet, which is at a steep incoming slope; this is consistent with the existing model and does not affect main channel results. The natural conditions 100-year velocity map and overbank velocity values can be found in Figure 64 and Table 19, respectively.

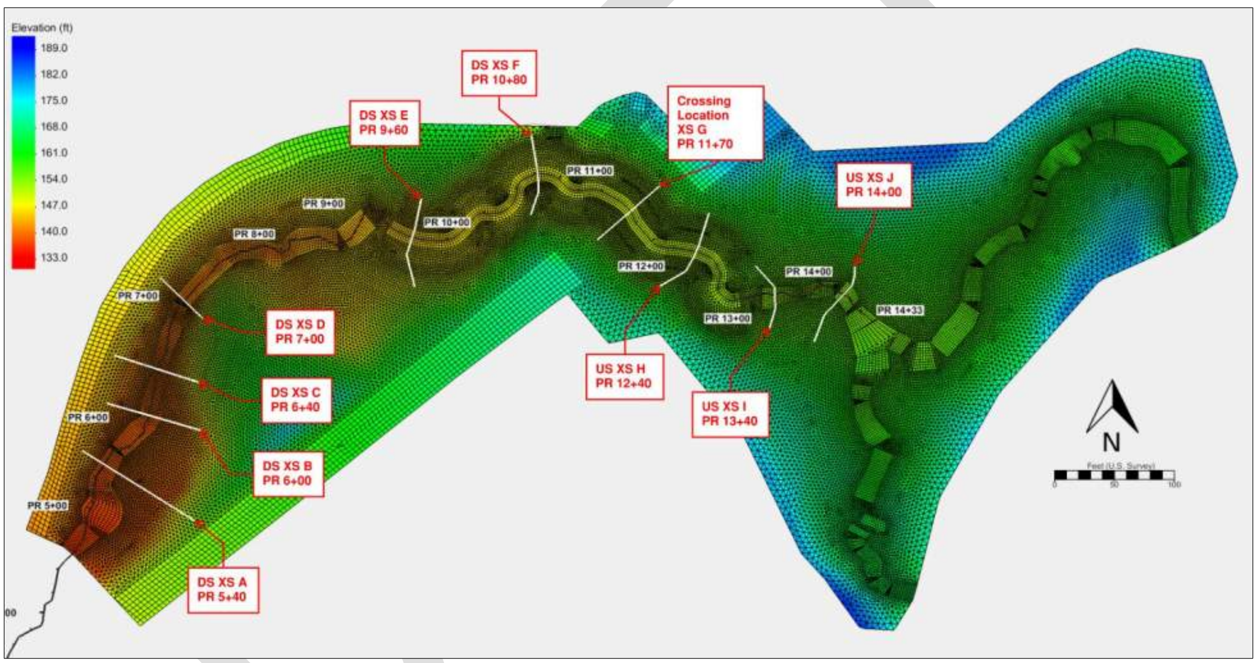

Figure 61. Locations of cross sections on proposed alignment used for results reporting

Table 18. Average main channel hydraulic results for natural conditions

Hydraulic parameter	Cross section	2-year	100-year	Projected 2080 100-year	500-year
Average WSE (ft)	DS PR 5+40 (A)	139.4	140.5	141.0	140.6
	DS PR 6+00 (B)	140.3	141.3	141.8	141.4
	DS PR 6+40 (C)	141.1	142.1	142.6	142.2
	DS PR 7+00 (D)	141.9	143.1	143.7	143.2
	DS PR 9+60 (E)	145.9	147.1	147.8	147.2
	DS PR 10+80 (F)	148.6	149.8	150.4	149.9
	Crossing Location 11+70 (G)	150.3	151.3	151.8	151.3
	US PR 12+40 (H)	151.6	152.8	153.3	152.8
	US PR 13+40 (I)	153.5	154.5	155.2	154.6
	US PR 14+00 (J)	154.4	155.8	156.4	155.9
Max depth (ft)	DS PR 5+40 (A)	1.8	2.9	3.4	3.0
	DS PR 6+00 (B)	1.4	2.4	2.8	2.4
	DS PR 6+40 (C)	1.8	2.9	3.3	2.9
	DS PR 7+00 (D)	1.8	3.0	3.6	3.1
	DS PR 9+60 (E)	1.6	2.8	3.4	2.8
	DS PR 10+80 (F)	1.7	2.9	3.5	3.0
	Crossing Location 11+70 (G)	1.6	2.6	3.1	2.6
	US PR 12+40 (H)	1.5	2.7	3.2	2.7
	US PR 13+40 (I)	1.6	2.6	3.2	2.6
	US PR 14+00 (J)	1.6	3.1	3.6	3.1
Average velocity (ft/s)	DS PR 5+40 (A)	3.6	4.9	5.2	5.0
	DS PR 6+00 (B)	3.1	4.8	5.6	4.9
	DS PR 6+40 (C)	3.3	5.6	6.6	5.7
	DS PR 7+00 (D)	3.0	5.5	6.6	5.7
	DS PR 9+60 (E)	4.0	5.4	5.5	5.4
	DS PR 10+80 (F)	3.8	5.0	5.2	5.0
	Crossing Location 11+70 (G)	3.7	5.3	5.6	5.3
	US PR 12+40 (H)	3.8	4.4	4.5	4.4
	US PR 13+40 (I)	2.6	5.2	5.6	5.2
	US PR 14+00 (J)	2.4	3.7	4.1	3.7
Average shear (lb/SF)	DS PR 5+40 (A)	1.3	1.7	1.7	1.7
	DS PR 6+00 (B)	1.1	1.8	2.2	1.8
	DS PR 6+40 (C)	0.9	2.0	2.6	2.1
	DS PR 7+00 (D)	0.8	1.9	2.6	2.0
	DS PR 9+60 (E)	1.4	1.9	1.9	1.9
	DS PR 10+80 (F)	1.2	1.6	1.7	1.6
	Crossing Location 11+70 (G)	1.2	2.0	2.1	2.0
	US PR 12+40 (H)	1.2	1.3	1.4	1.3
	US PR 13+40 (I)	0.6	1.8	2.0	1.8
	US PR 14+00 (J)	0.6	0.9	1.1	0.9

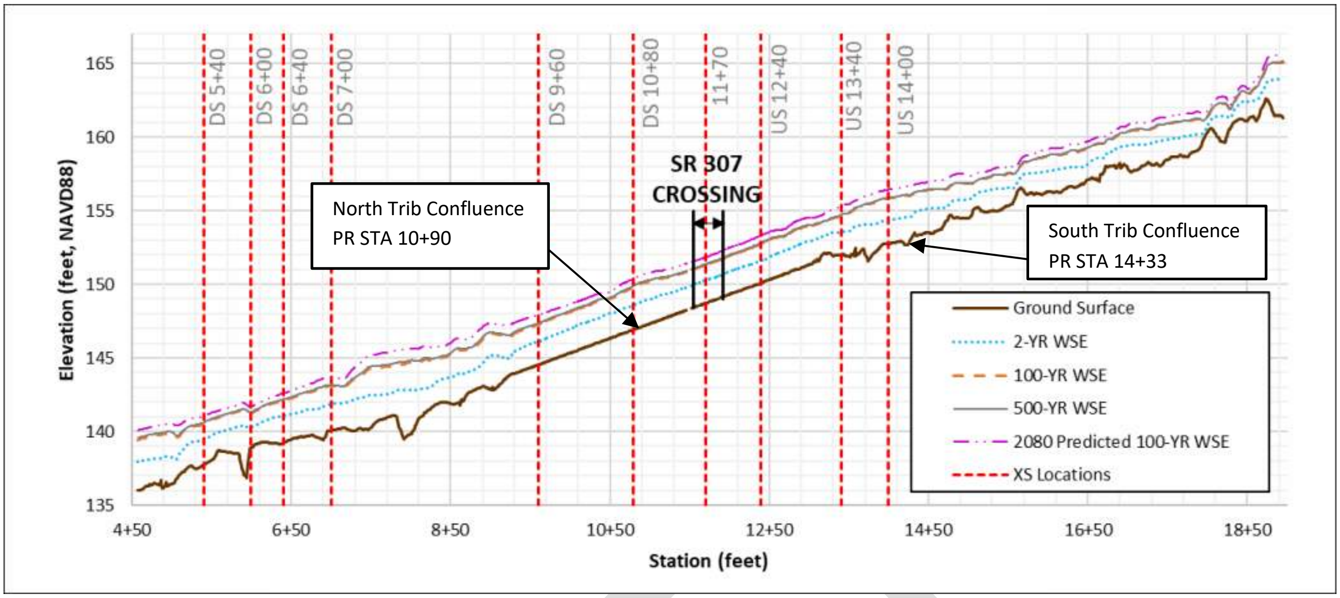

Figure 62. Natural-conditions water surface profiles

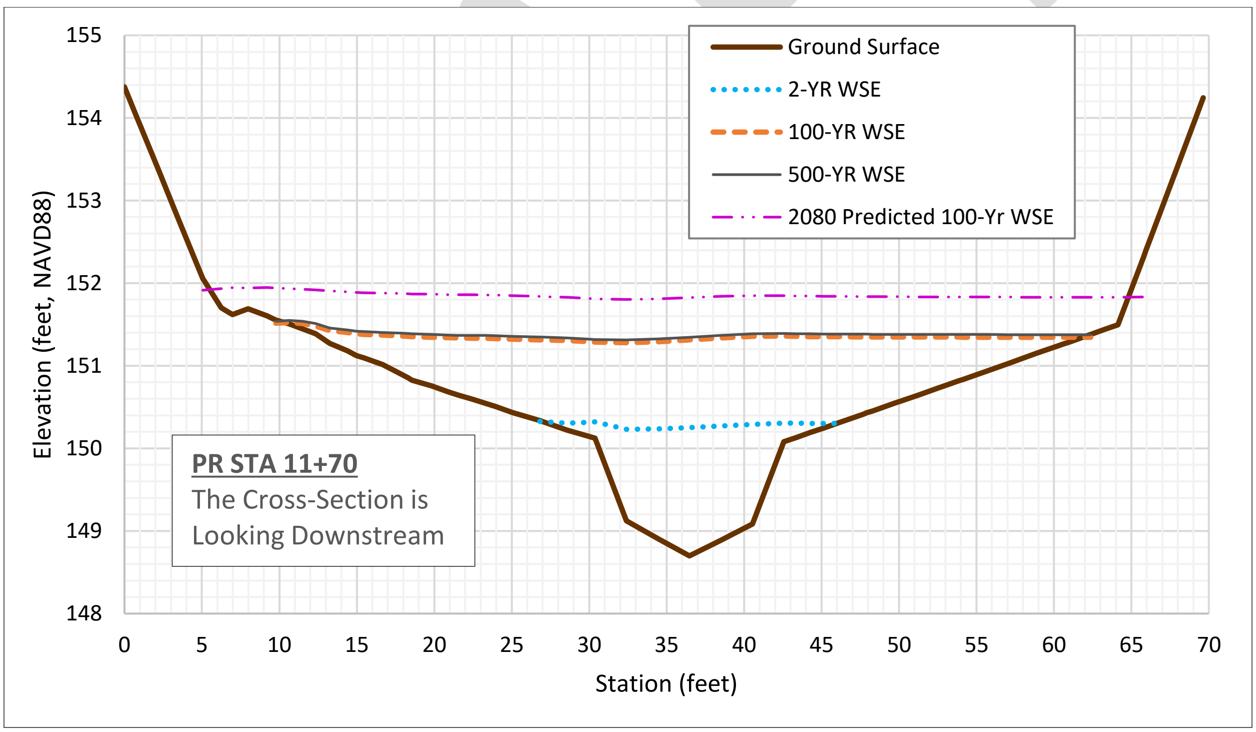

Figure 63. Typical section through crossing location (STA 11+70)

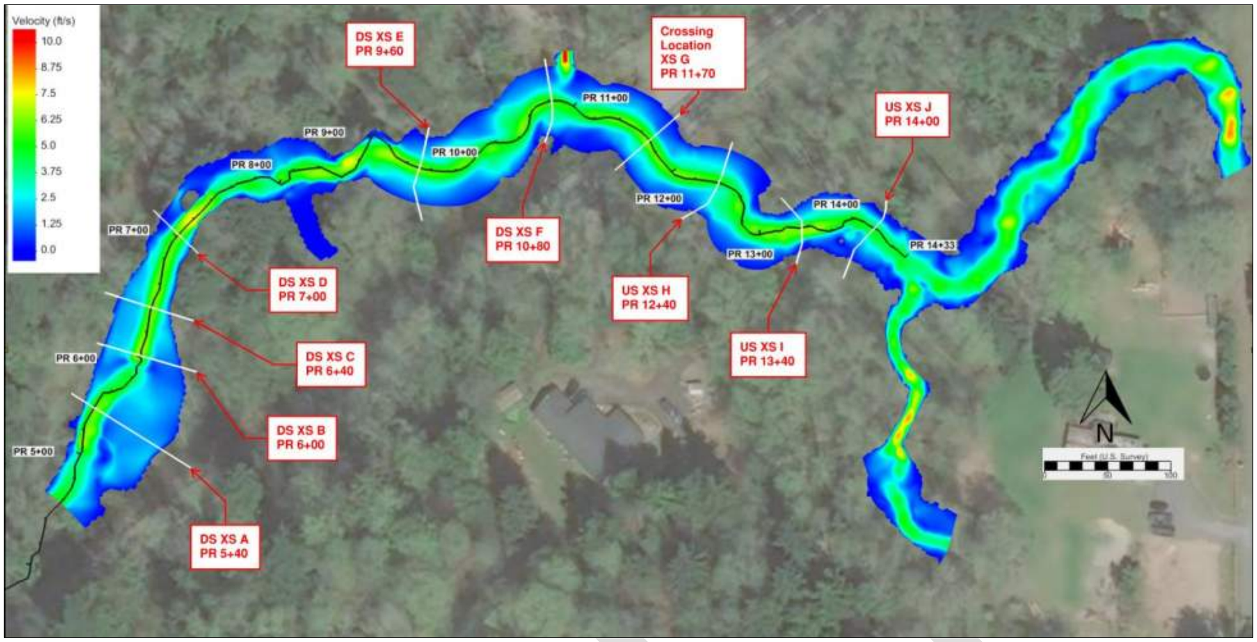

Figure 64. Natural-conditions 100-year velocity map

Table 19. Natural-conditions average channel and floodplains velocities

Cross-section location	Q100 average velocities (ft/s)			2080 projected Q100 average velocity (ft/s)		
	LOB[a]	Main channel	ROB[a]	LOB[a]	Main channel	ROB[a]
DS PR 5+40 (A)	1.5	4.9	2.2	2.1	5.2	2.6
DS PR 6+00 (B)	1.0	4.8	2.1	1.6	5.6	2.9
DS PR 6+40 (C)	2.0	5.6	1.8	3.1	6.6	2.8
DS PR 7+00 (D)	2.2	5.5	0.7	3.1	6.6	1.8
DS PR 9+60 (E)	2.0	5.4	1.7	2.0	5.5	1.9
DS PR 10+80 (F)	2.5	5.0	1.4	3.8	5.2	1.7
Crossing Location 11+70 (G)	2.3	5.3	0.9	3.3	5.6	1.2
US PR 12+40 (H)	1.9	4.4	2.0	2.9	4.5	2.4
US PR 13+40 (I)	0.5	5.2	1.8	1.6	5.6	2.4
US PR 14+00 (J)	1.4	3.7	0.3	2.0	4.1	0.7

Right overbank (ROB)/left overbank (LOB) locations were approximated by topographic grade breaks.

5.4 Proposed Conditions: 25-foot Minimum Hydraulic Width

The hydraulic width is defined as the width perpendicular to the creek beneath the proposed structure that is necessary to convey the design flow and allow for natural geomorphic processes. The hydraulic modeling assumes vertical walls at the edge of the minimum hydraulic width unless otherwise specified. See Section 4.2.2 for a description of how the minimum hydraulic width was determined.

The proposed conditions SRH-2D model results were used to evaluate the hydraulic conditions within the proposed crossing that has a 25-foot-wide hydraulic width for the 2-, 100-, 2080 100-, and 500-year peak flood events for Northeast Dogfish Creek at the project site. Inundation extents and results extracted at selected cross-section locations are discussed in the following paragraphs. Appendix H contains additional cross-sectional plots as well as plan view figures of hydraulic modeling results. The proposed conditions model results extracted from the selected cross-sections A through J are shown in Table 20. The locations of the cross-sections along the proposed alignment are shown in Figure 65. Cross-sections in the proposed conditions model are located at the same place as they are in the natural conditions model. Model results show no roadway overtopping in the proposed conditions. The backwatering shown at the culvert inlet in the existing conditions model is eliminated in the proposed conditions.

Due to the unconfined nature of the channel and 51.8-foot 100-year flow width through the crossing demonstrated in the natural conditions results, high flow events would make contact with the walls of a 25-foot-wide structure. Modeling results suggest that the 100-, 500-, and 2080 projected 100-year events would all reach the side walls, with maximum depths up the side of the structure walls of 1.1, 1.1, and 1.8 feet, respectively. Utilizing the unconfined design methodology, the velocity ratio comparing proposed velocity through the proposed structure (cross-section G) to natural conditions velocity directly upstream of the regraded channel tie-in (cross-section I) was determined to be 1.1 (see Section 4.2.2 for full analysis). Alternating meander bars are proposed within the structure to prevent entrainment of flow along the edge of the structure and maintain a planform sinuosity through the crossing. The proposed channel has roughness elements added to it which match expected natural conditions. Meander bars are included in the sinuous channel designed in the proposed surface.

Figure 66 shows the proposed WSE maintains a 100-year flow depth of approximately 1.0 to 5.2 feet. The 100-year model results within the structure show flow contacting both ends of the 25-foot-wide opening. The 2-year flow width within the structure is 18.9 feet (Figure 67). The 2-year flow is nearly contained within the main channel, while the 100-year flow overtops the main channel. The 2-year flow width within the reference reach at EX STA 6+00 is 17.9 feet while being close to the bankfull channel elevation (Figure 60). This is a consistent trend at all cross sections taken throughout the modeled domain and is similar to modeling results for existing conditions (Appendix H).

Main channel average velocity hovers consistently around 3.6 to 5.6 feet per second throughout the modeled domain during the 100-year flow event, and 4.1 to 6.6 feet per second during the 2080 projected 100-year event. The floodplain velocities are relatively slow, ranging from 0.3 to 2.9 feet per second during the 100-year event and 0.6 to 4.6 feet per second during the 2080 projected 100-year event. The maximum 100-year velocity of 14.6 feet per second occurs at the north tributary basin inlet, which is at a steep incoming slope; this is consistent with the existing and natural models and does not affect main channel results. The proposed conditions 100-year velocity map and overbank velocity values can be found in Figure 68 and Table 21. Slightly higher shear stress are observed at the bends downstream of the crossing, but no extremely high shear stress are found in the proposed conditions model. Velocity values in the existing conditions reference reach are within 0.3 feet per second of the proposed velocity values at the same locations.

During the 100-, 500-, and 2080 100-year flow events, the abandoned channel near the existing culvert inlet becomes inundated with low velocity flow (Figure 68 US PR 12+40). This will be left in the proposed design surface as a habitat feature. As it only becomes inundated during the 100-year event, it is expected to remain dry or become disconnected off-channel habitat for amphibians or birds.

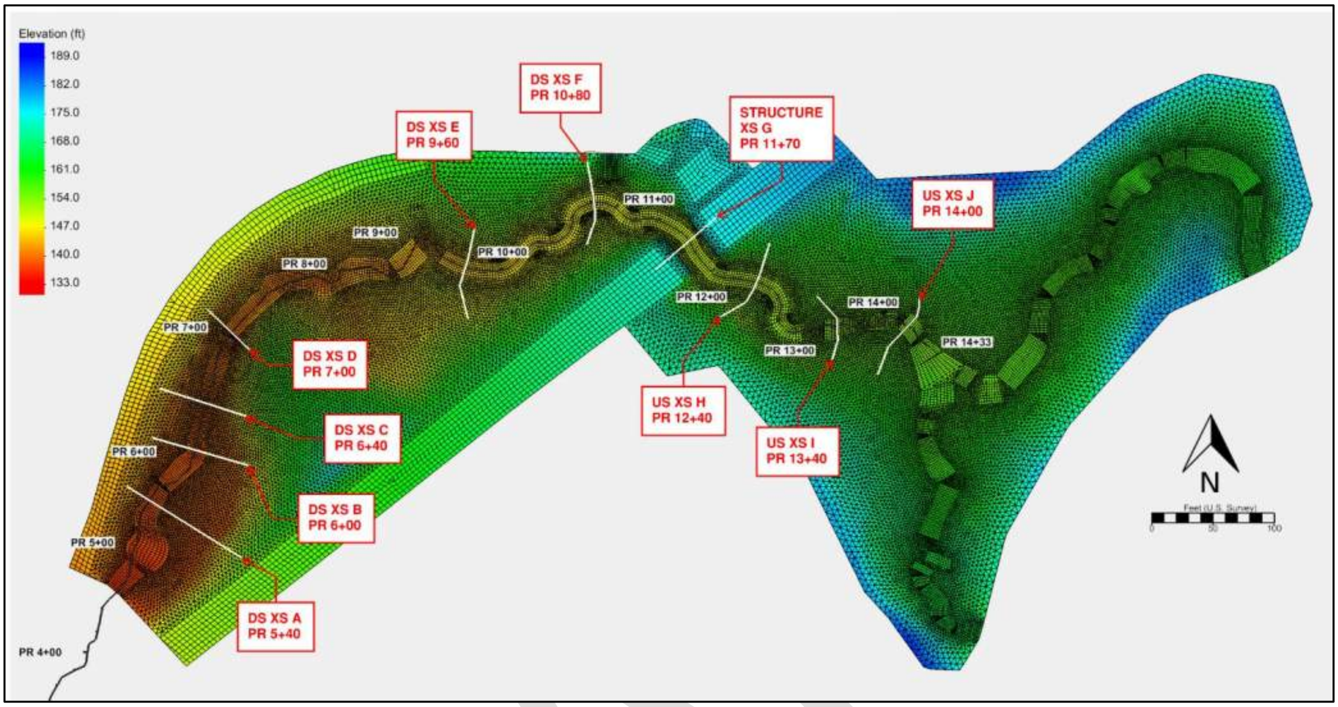

Figure 65. Locations of cross sections on proposed alignment used for results reporting

Table 20. Average main channel hydraulic results for proposed conditions

Hydraulic parameter	Cross section	2-year	100-year	2080 Projected 100-year	500-year
Average WSE (ft)	DS PR 5+40 (A)	139.4	140.5	141.0	140.6
	DS PR 6+00 (B)	140.3	141.3	141.8	141.4
	DS PR 6+40 (C)	141.1	142.1	142.6	142.2
	DS PR 7+00 (D)	141.9	143.1	143.7	143.2
	DS PR 9+60 (E)	145.9	147.2	147.9	147.3
	DS PR 10+80 (F)	148.6	150.0	150.8	150.1
	Structure 11+70 (G)	150.3	151.4	152.1	151.5
	US PR 12+40 (H)	151.6	152.8	153.6	152.8
	US PR 13+40 (I)	153.5	154.6	155.3	154.7
	US PR 14+00 (J)	154.3	155.8	156.4	155.9
Max depth (ft)	DS PR 5+40 (A)	1.8	2.9	3.4	3.0
	DS PR 6+00 (B)	1.4	2.4	2.8	2.4
	DS PR 6+40 (C)	1.8	2.9	3.3	2.9
	DS PR 7+00 (D)	1.8	3.0	3.6	3.1
	DS PR 9+60 (E)	1.6	2.8	3.6	2.9
	DS PR 10+80 (F)	1.7	3.1	3.8	3.2
	Structure 11+70 (G)	1.6	2.7	3.4	2.8

Hydraulic parameter	Cross section	2-year	100-year	2080 Projected 100-year	500-year
	US PR 12+40 (H)	1.5	2.7	3.5	2.7
	US PR 13+40 (I)	1.6	2.7	3.4	2.7
	US PR 14+00 (J)	1.6	3.1	3.6	3.1
Average velocity (ft/s)	DS PR 5+40 (A)	3.6	4.9	5.2	5.0
	DS PR 6+00 (B)	3.1	4.8	5.6	4.9
	DS PR 6+40 (C)	3.3	5.6	6.6	5.7
	DS PR 7+00 (D)	3.0	5.5	6.6	5.7
	DS PR 9+60 (E)	4.0	5.6	6.1	5.7
	DS PR 10+80 (F)	3.9	4.8	4.9	4.8
	Structure 11+70 (G)	3.7	5.5	6.1	5.5
	US PR 12+40 (H)	3.6	5.1	5.0	5.1
	US PR 13+40 (I)	2.6	4.9	5.2	5.0
	US PR 14+00 (J)	2.5	3.6	4.1	3.6
Average shear (lb/SF)	DS PR 5+40 (A)	1.3	1.7	1.7	1.7
	DS PR 6+00 (B)	1.1	1.8	2.2	1.8
	DS PR 6+40 (C)	0.9	2.0	2.7	2.1
	DS PR 7+00 (D)	0.8	1.9	2.5	2.0
	DS PR 9+60 (E)	1.4	2.1	2.3	2.2
	DS PR 10+80 (F)	1.2	1.5	1.5	1.5
	Structure 11+70 (G)	1.2	2.0	2.4	2.0
	US PR 12+40 (H)	1.2	1.8	1.6	1.8
	US PR 13+40 (I)	0.6	1.6	1.7	1.6
	US PR 14+00 (J)	0.6	0.9	1.0	0.9

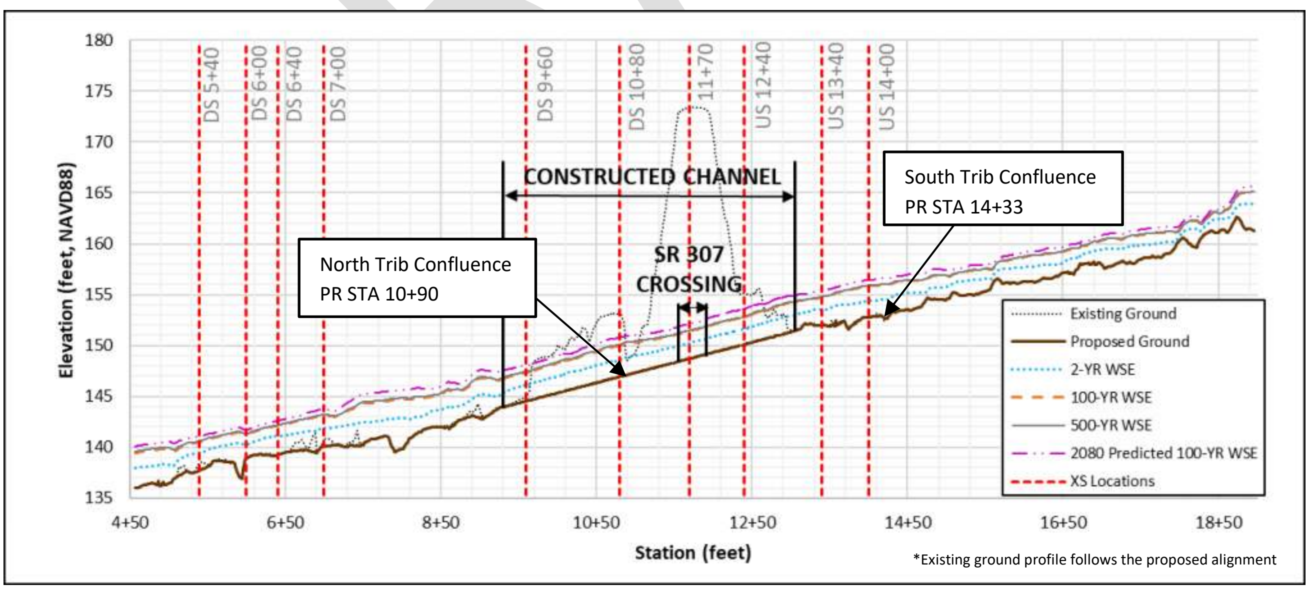

Figure 66. Proposed-conditions water surface profiles

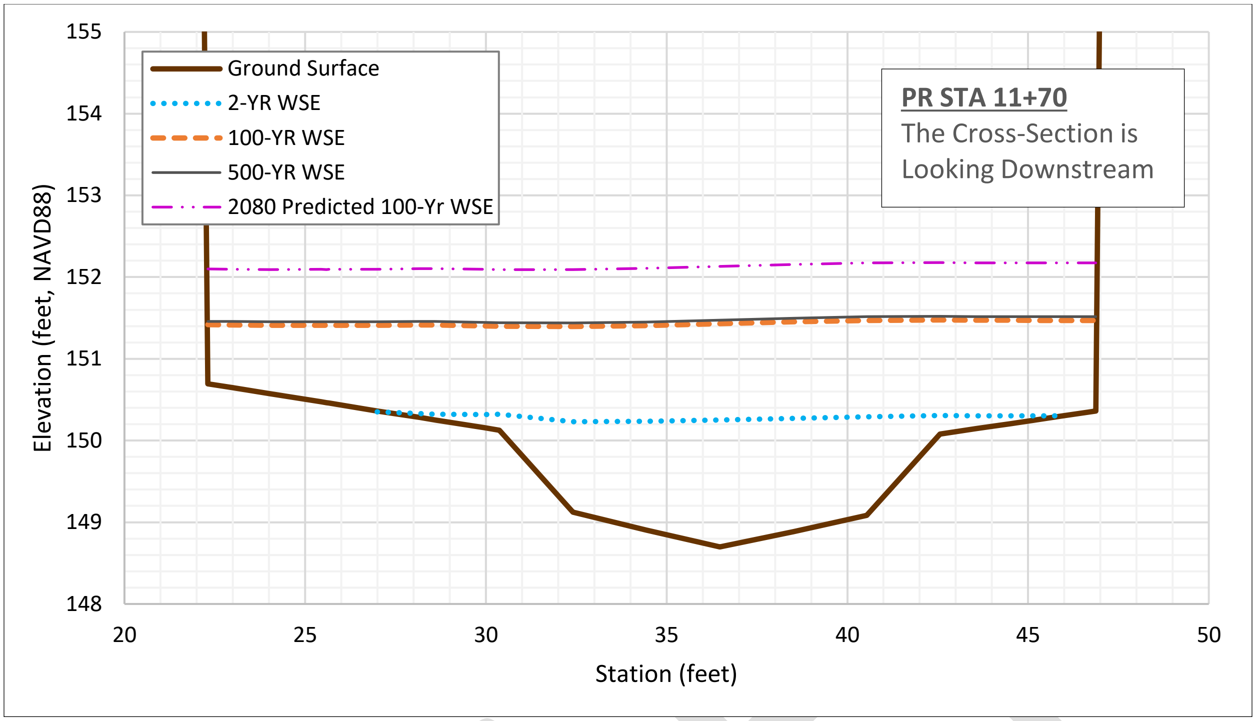

Figure 67. Typical section through proposed structure (STA 11+70)

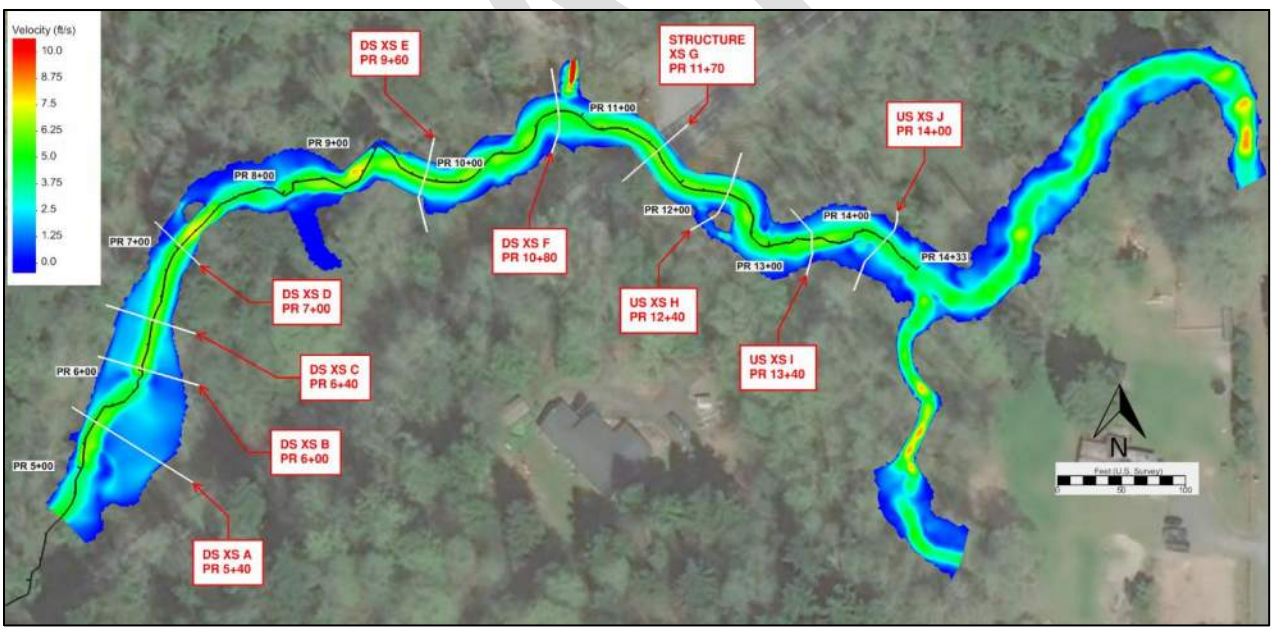

Figure 68. Proposed-conditions 100-year velocity map

Table 21. Proposed-conditions average channel and floodplains velocities

Cross-section location	Q100 average velocities (ft/s)			2080 Projected Q100 average velocity (ft/s)		
	LOB[a]	Main channel	ROB[a]	LOB[a]	Main channel	ROB[a]
DS PR 5+40 (A)	1.5	4.9	2.2	2.1	5.2	2.7
DS PR 6+00 (B)	1.0	4.8	2.1	1.6	5.6	2.9
DS PR 6+40 (C)	2.0	5.6	1.8	3.1	6.6	2.7
DS PR 7+00 (D)	2.3	5.5	0.7	2.8	6.6	1.8
DS PR 9+60 (E)	2.9	5.6	2.5	2.0	6.1	2.6
DS PR 10+80 (F)	2.8	4.8	1.5	3.5	4.9	1.4
Structure 11+70 (G)	3.4	5.5	1.9	4.6	6.1	2.6
US PR 12+40 (H)	0.8	5.1	3.2	1.3	5.0	3.4
US PR 13+40 (I)	0.4	4.9	1.6	1.4	5.2	2.4
US PR 14+00 (J)	1.4	3.6	0.3	1.9	4.1	0.6

Right overbank (ROB)/left overbank (LOB) locations were approximated by topographic grade breaks.

6 Floodplain Evaluation

This project is within a FEMA special flood hazard area (SFHA) Zone X; see Appendix A for FIRMette. The existing-project and expected proposed-project conditions were evaluated to determine whether the project would cause a change in flood risk.

6.1 Water Surface Elevations

The existing model results show heavy backwatering during the 100- and 500-year storm events (Figure 57), which poses a risk to roadways and properties near the existing culvert inlet in the case of a flood. With the removal of the undersized culvert, the proposed model eliminates the upstream backwatering and significantly reduces the water surface elevations in the upstream reach. The 100-year existing and proposed WSEs converge approximately 650 feet upstream of the existing culvert inlet, which is the extent of the existing conditions backwatering (Figure 69). The existing and proposed WSEs are largely similar downstream of the culvert outlet, with the exception of several minor bumps occurring in the existing water surface profile that are removed in the proposed conditions profile due to uniform ground surface grading.

A flood risk assessment will be developed during later stages of the design.

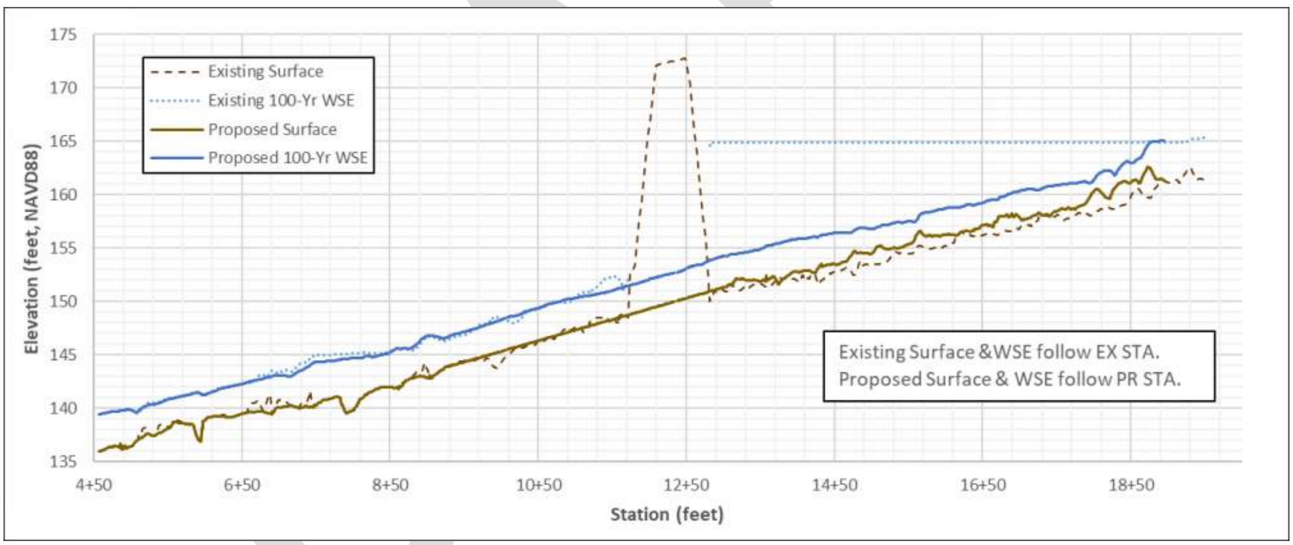

Figure 69. Existing- and proposed-conditions 100-year water surface profile comparison along proposed alignment

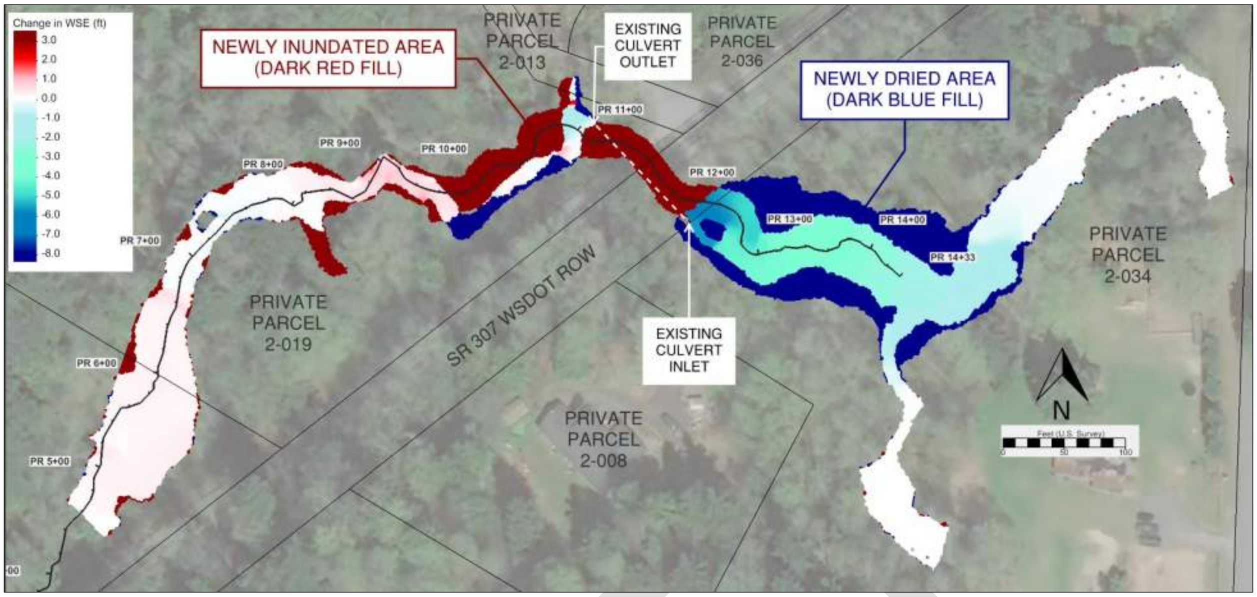

Figure 70. 100-year WSE change from existing to proposed conditions

7 Preliminary Scour Analysis

For this preliminary phase of the project, the risk for lateral migration, potential for long-term degradation and evaluation of preliminary total scour is based on available data, including but not limited to LiDAR data (DNR 2018), a ground survey and a preliminary geotechnical scoping document completed by WSDOT as part of this project (Section 2.7.1). This evaluation is to be considered preliminary and is not to be taken as a final recommendation.

Using the results of the hydraulic analysis (Section 5.4), based on the recommended minimum hydraulic opening (25 feet), and considering the potential for lateral channel migration, preliminary scour calculations for the scour design flood and scour check flood (both the 2080 projected 100-year, 311.5 cfs) were performed following the procedures outlined in *Evaluating Scour at Bridges, HEC No. 18* (Arneson, et al. 2012). Scour components considered in the analysis include:

- Long-term degradation
- Contraction scour
- Local scour

In addition to the three scour components listed above, the potential for lateral migration was assessed to evaluate total scour at the proposed highway infrastructure. These various scour components will be discussed in the following sections.

At the Project Crossing the 2080 projected 100-year flow max depth is 3.43 feet through the crossing and required maintenance clearance is 10 feet. There is approximately 18 feet of road fill above the crossing and the proposed design is expected to be able to accommodate the 2080 projected 100-year flow.

7.1 Lateral Migration

A Geotechnical scoping memo has been completed for this crossing. At the time of writing, lateral migration is assumed to be not low due to high energy of system and no geotechnical information suggesting the channel is restricted from migrating. The channel is an unconfined system (Section 2.7.2.1) and the floodplains are inundated at the 2-year event (Section 5.2). The channel location shifts laterally across the valley bottom resulting in some sinuosity in the existing condition (Section 4.1.2). No obvious signs of recent channel migration were observed and the lateral complexity that was seen appeared to be forced by LWM and debris jams (Section 2.7.2). No freshly eroded banks were observed nor were there any developed floodplain channels. The potential risk for channel migration is based on the valley scale channel type. Lateral complexity was built into the design using meander bars, habitat boulders and LWM (Section 4.3). No scour countermeasures are recommended; however, they may become necessary as the project progresses and the structure size, type, and location are known.

7.2 Long-term Degradation of the Channel Bed

Long-term degradation for Northeast Dogfish Creek at the project crossing was estimated based on site visit observation, watershed assessment, LiDAR and survey profile, and the geotechnical scoping memo. There is one knickpoint which was observed approximately 300 feet downstream of Site ID 991999 (Figure 71). A 3-foot vertical drop in the channel bed was observed in December of 2021 (Section 2.7.4). The drop was covered in woody debris and live roots. This drop is the most pronounced discontinuity in the slope of the streambed, and is assumed to be the most likely point of instability. The stability or rate of erosion at this point is not known, but there is potential that, in the long term, the base of the drop may become the new base elevation controlling the slope and elevation of the stream (Figure 71). The geotechnical scoping memo indicates the soils at the elevation of the streambed are coarse grained glacial deposits with a HEC-18 erodibility of Medium (III). The geotechnical scoping memo indicates there is no bedrock layer to limit long-term degredation.

To assess the magnitude of potential long-term degradation if the knickpoint were to migrate upstream, the vertical drop of the knickpoint was projected back to the crossing. Visually comparing thalweg elevations from the LiDAR and the ground survey revealed a constant offset between the two datasets. A 1-foot vertical adjustment was applied to the LiDAR and equilibrium slope (Section 2.7.4) elevations so they would more closely match the ground survey. The adjusted equilibrium slope and proposed surface cause the bed elevation at the crossing (STA 12+20) to be 149.7 and 149.9 feet, respectively. It was assumed the streambed would stabilize at the equilibrium slope after degradation had migrated upstream from the knickpoint. No catch point is noted as there is no known hardpoint to arrest the theoretical erosion upstream. This analysis suggests if the existing 3-foot knickpoint were to migrate upstream, it would result in 3 feet of degradation at the crossing.

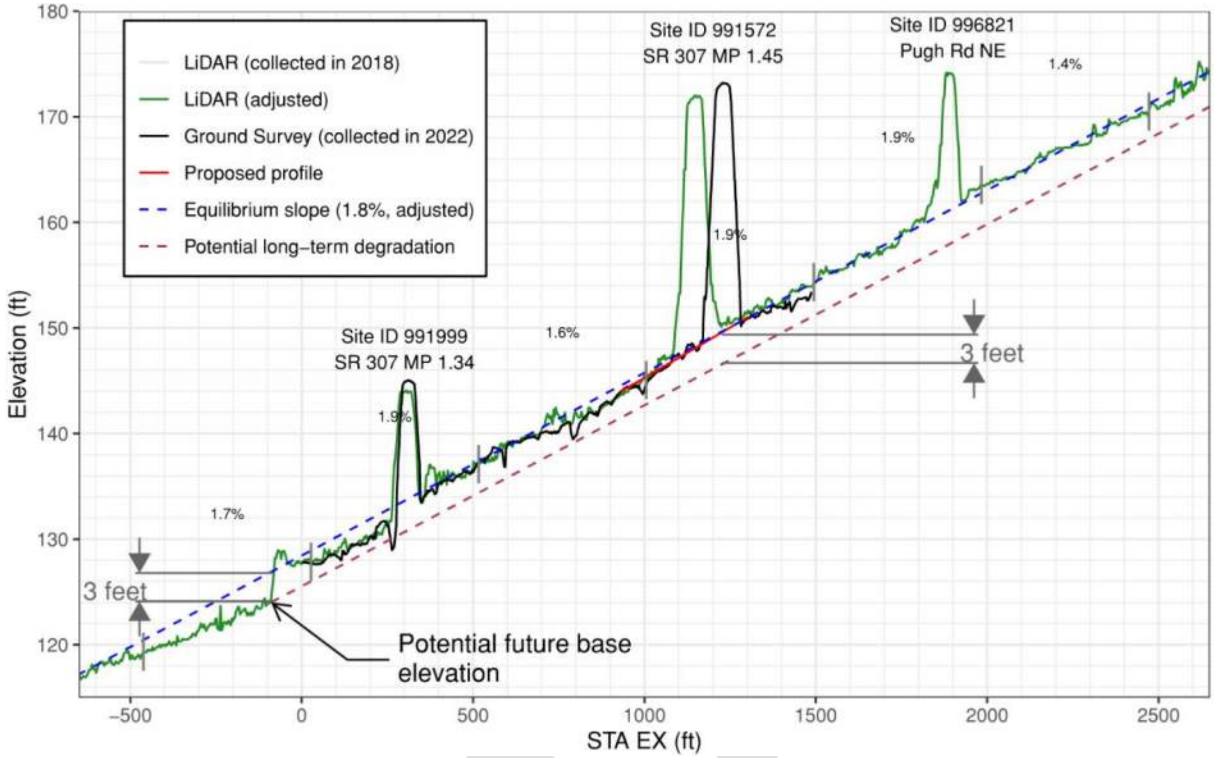

Figure 71. Potential long-term degradation at the proposed structure upstream face. Note the LiDAR and survey thalweg alignments differ distorting the relative location of the Project Crossing farther from STA 0+00.

7.3 Contraction Scour

The contraction scour for the project crossing was estimated following the methodology outlined in Chapter 6 of HEC-18 (Arneson, et al. 2012). This estimation used the model results extracted from SRH-2D (Aquaveo 2021) and the Bridge Scour Analysis tool in Hydraulic Toolbox for calculation (FHWA 2021). The critical velocity of the proposed D_{50} is calculated and compared to the average velocity upstream of the structure to determine the scour condition, live-bed or clear-water, that exists at the crossing.

The 2-year, 10-year, 25-year, 50-year, 100-year, 2080 projected 100-year and 500-year proposed conditions model results with the 25-foot MHO described in Section 5.4 were used for the analysis. Critical velocity index (CVI) maps were developed for the 2-year and 100-year events, both of which showed average velocities through the crossing lower than the calculated critical velocity. This task was also performed for the 2080 projected 100-year event, which showed average velocities through the main channel of the crossing higher than the calculated critical velocity. Values of the CVI were not significantly greater than 1.0 and values above 1.0 had a limited spatial extent. Due to this, the 2080 projected 100-year event was considered clearwater and bank arcs were not adjusted to follow the boundaries of the areas where the CVI was greater than one. The results of the analysis estimate 0.0 feet of contraction scour among all the simulated events. See Appendix K for detailed contraction scour calculations and critical velocity index maps.

7.4 Local Scour

The following sections described the scour methodology and results of the local scour components.

7.4.1 Pier Scour

The crossing will not have piers and therefore pier scour was not calculated.

7.4.2 Abutment Scour

Abutment scour was estimated using the National Cooperative Highway Research Program (NCHRP) 24-20 approach for the scour design flood and scour check flood. Assuming the most conservative scenario-when vertical abutment walls are constructed immediately on two sides of the 25-foot-wide MHO-the 100-year flow will engage the abutment structure at a depth 1.0 foot on the floodplain benches. The scour calculation was completed using the Abutment Scour tool in Hydraulic Toolbox. The 2080 projected 100-year flow is the scour design and the scour check flood based on the results from the Hydraulic Toolbox detailed in Appendix K. It is the design flood because it results in greater scour than the 100 year flow and it is the check flood due to resulting in greater scour and having greater flow than the 500-year flow. The maximum depth of scour computed in the NCHRP method is 1.4 feet for the 2080 projected 100-year flow. This assessment is specific to the 25-foot MHO that is currently proposed by PACE. Abutment scour should be re-evaluated when a structure type is recommended for this crossing by WSDOT in later stages of the design (Section 4.2.6).

The risk of lateral migration for this crossing is assumed as not low risk at this current stage of design. The abutment scour will be applied to the thalweg elevation in accounting for total scour to provide a conservative scour depth estimate. This approach assumes that the thalweg can migrate to either abutment during the design life of the structure.

7.4.3 Bend Scour

Bend scour was calculated following the methodology outlined in HEC-23 (Lagasse et al. 2012). Depth of bend scour was estimated using Maynord’s method, the design flood and the check flood (2080 projected 100-year flood). This method is based on empirical studies conducted in sand bed systems and will provide conservative estimates for gravel bed streams. The Thorne equation as well as the National Engineering Handbook method were considered. However the radius of curvature versus the top width of the channel is out of range for the Thorne equation and the National Engineering Handbook method is a check for the other methods that gives a highly conservative answer, not appropriate for the crossing.

Average flow depth was measured over the bankfull channel where the bend scour will occur. The analysis indicates that the depth of bend scour is 2.7 feet at the design flood and at the check flood.

7.5 Total Scour

Calculated total depths of scour for the scour design flood and scour check flood at the proposed NE Dogfish Creek SR 307 crossing are provided in Table 22. The total scour of the project crossing is evaluated up to 2080 projected 100-year flow. Contraction scour in Table 22 is not added to the total because local scour estimated by NCHRP method includes contraction scour in its estimate, and the two are not additive. These preliminary recommendations could change as the design progresses and should be reevaluated during later stages of design.

Table 22. Scour Analysis Summary

Calculated Scour Components and Total Scour for SR 307 MP 1.45 NE Dogfish Creek		
	Scour design flood (2080 projected 100-year flow, 311.5 cfs)	**Scour check flood (2080 projected 100-year flow, 311.5 cfs)**
Long-term degradation (ft)	3.0	3.0
Contraction scour (ft)	0.0	0.0
Bend scour (ft)	2.7	2.7
Abutment scour (ft)[a]	1.4	1.4
Total depth of scour (ft)[b]	**7.1**	**7.1**

a. *Abutment scour is estimated using Abutment Scour Tool in Hydraulic Toolbox with method outlined in NCHRP 24-20, which include contraction scour in their estimates.*

b. *Total scour includes long-term degradation plus contraction or local scour, whichever is greater. Depth of total scour is applied to the thalweg elevation of the proposed channel to determine the total scour elevation at each infrastructure component*

8 Scour Countermeasures

For the proposed crossing the estimated total depth of scour during the scour design flood and scour check flood is 5.7 feet. Assuming all structure and foundation walls including any abutment wall, wing walls, and retaining walls extend below the scour design flood total depth of scour and any foundation caps and footing extend below the check flood total depth of scour then scour countermeasures are not necessary for the project crossing. If the structure walls do not extend below the elevation associated with the total depth of scour, then the need for scour countermeasures will be reevaluated in a later stage of design once structure and foundation designs are determined.

The likelihood of scour countermeasures increases if a full span bridge is the selected structure. Elements of a water crossing that may need a scour countermeasure include but are not limited to the bridge substructure, walls, and the roadway embankment. Structural foundations cannot rely on scour countermeasure for the integrity of the structure. If scour countermeasures are deemed necessary, they will not encroach within the structure free zone unless there has been additional coordination and acceptance from WDFW and Tribes.

9 Summary

Table 23 presents a summary of the results of this PHD Report.

Table 23. Report summary

Stream crossing category	Element	Value	Report location
Habitat gain	Total length	7,175 LF	2.1 Site Description
Bankfull width	Reference reach found?	Yes	2.7.1 Reference Reach Selection
	Design BFW	12 ft	2.7.2 Channel Geometry
	Concurrence average BFW	12.4 ft	2.7.2 Channel Geometry
Floodplain utilization ratio (FUR)	Flood-prone width	42.9 ft	2.7.2.1 Floodplain Utilization Ratio
	Average FUR	3.5	2.7.2.1 Floodplain Utilization Ratio
Channel morphology	Existing	See link	2.7.2 Channel Geometry
	Proposed	See link	4.3.2 Channel Complexity
Hydrology/design flows	100 yr flow	193.5 cfs	3 Hydrology and Peak Flow Estimates
	2080 projected 100 yr flow	311.5 cfs	3 Hydrology and Peak Flow Estimates
	2080 100 yr used for design	No	3 Hydrology and Peak Flow Estimates
	Dry channel in summer	No	3 Hydrology and Peak Flow Estimates
Channel geometry	Existing	See link	2.7.2 Channel Geometry
	Proposed	See link	4.1.1 Channel Planform and Shape
Channel slope/gradient	Existing culvert	1.4%	2.6.2 Existing Conditions
	Reference reach	1.7%	2.7.1 Reference Reach Selection
	Proposed	2.0%	4.1.3 Channel Gradient
Hydraulic width	Existing	4 ft	2.6.2 Existing Conditions
	Proposed	25 ft	4.2.2 Hydraulic Width
	Added for climate resilience	No	4.2.2 Hydraulic Width
Vertical clearance	Required freeboard	3 ft	4.2.3 Vertical Clearance
	Required freeboard applied to 100 yr or 2080 100 yr	100 yr	4.2.3 Vertical Clearance
	Maintenance clearance	Required 10 ft	4.2.3 Vertical Clearance
	Low chord elevation	See link	4.2.3 Vertical Clearance
Crossing length	Existing	110.4 ft	2.6.2 Existing Conditions
	Proposed	36 ft	4.2.4 Hydraulic Length
Structure type	Recommendation	No	4.2.6 Structure Type
	Type	NA	4.2.6 Structure Type
Substrate	Existing	See link	2.7.3 Sediment
	Proposed	See link	4.3.1 Bed Material
	Coarser than existing?	No	4.3.1 Bed Material
Channel complexity	LWM for bank stability	No	4.3.2 Channel Complexity
	LWM for habitat	Yes	4.3.2 Channel Complexity
	LWM within structure	No	4.3.2 Channel Complexity
	Meander bars	2	4.3.2 Channel Complexity
	Boulder clusters	0	4.3.2 Channel Complexity
	Coarse bands	0	4.3.2 Channel Complexity

Stream crossing category	Element	Value	Report location
	Mobile wood	No	4.3.2 Channel Complexity
Floodplain continuity	FEMA mapped floodplain	No	0 Floodplain Evaluation
	Lateral migration	Yes	2.7.5 Channel Migration
	Floodplain changes?	No	0 Floodplain Evaluation
Scour	Analysis	See link	7
	Scour countermeasures	Determined at FHD	0 Scour Countermeasures
Channel degradation	Potential?	Yes	7.2 Long-term Degradation of the Channel Bed
Channel degradation	Allowed?	Yes	7.2 Long-term Degradation of the Channel Bed

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Appendices

Appendix A: FEMA Floodplain Map

Appendix B: Hydraulic Field Report Form

Appendix C: Streambed Material Sizing Calculations

Appendix D: Stream Plan Sheets, Profile, Details

Appendix E: Manning's Calculations

Appendix F: Large Woody Material Calculations

Appendix G: Future Projections for Climate-Adapted Culvert Design

Appendix H: SRH-2D Model Results

Appendix I: SRH-2D Model Stability and Continuity

Appendix J: Reach Assessment

Appendix K: Scour Calculations

Appendix L: Floodplain Analysis (FHD ONLY)

Appendix M: Scour Countermeasure Calculations (FHD ONLY)

Appendix A: FEMA Floodplain Map

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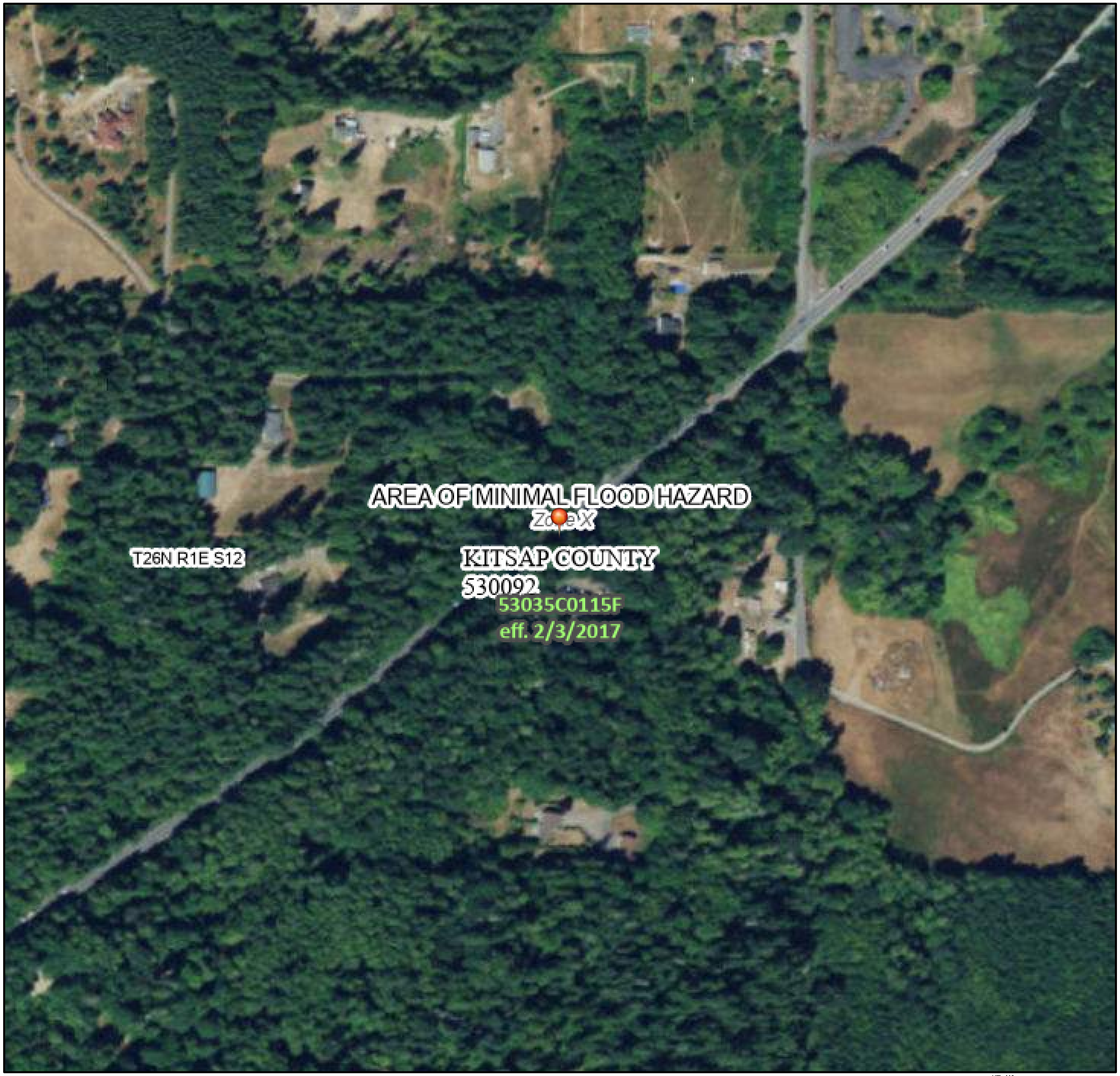


Appendix B: Hydraulic Field Report Form

WSDOT Hydraulics Section	Hydraulics Field Report	Project Number: Y-12554 Task AC
	Project Name: SR 307 MP1.45 Unnamed to Dogfish Ck 991572	Date: December 2, 2021 December 9, 2021
	Project Office:	Time of Arrival: 2: 2:45 pm 9: 4:00 pm
	Stream Name: Unnamed	Time of Departure: 2: 4:15 pm 9: 4:30 pm
WDFW ID Number: 991572	Tributary to: Dogfish Creek	Weather: Dec 2: overcast with occasional rain, ~ 45° F Dec 9: overcast, ~45° F
State Route/MP: SR 307 MP 1.45	Township/Range/Section/ ¼ Section: T26N R1E S12 SENW1/4	Prepared By: C. Nicol, H. Moen
County: Kitsap	Purpose of Site Visit: Identify reference reach and collect stream channel measurements	WRIA: 15.0286

Meeting Location:
Dec. 2: Walmart at 21200 Olhava Way NW, Poulsbo, WA 98370 parking lot
Dec. 9: Petco, 9589 Ridgetop Blvd NW, Silverdale, WA 98383

Attendance List:

Name	Organization	Role
Shane Sheldon (Dec. 2 and 9)	PACE	Lead Engineer
Colin Nicol (Dec. 2 and 9)	PACE	Environmental Scientist
Tasha Wang (Dec. 2	PACE	Project Engineer
Miranda Smith (Dec. 9)	PACE	Project Manager
Taryn Mulvihill (Dec. 9)	PACE	Junior Engineer
Henry Moen (Dec. 2)	PACE	E.I.T.

Dogfish Creek Unnamed Tributary – State Route (SR) 307 Crossing Site ID: 991572 (Crossing 991572) has been identified as a fish passage barrier by Washington Department of Fish and Wildlife (WDFW). P ACE is working with the Washington State Department of Transportation (WSDOT) to complete a preliminary design for a fish-passable crossing. The following Hydraulics Field Report documents the geomorphic, biological, and hydraulic field assessment of Crossing 991572 conducted by PACE. The reaches both upstream and downstream of the crossing, can be accessed directly from SR 307 by parking in a private driveway nearly directly above the nearby crossing of 930880. PACE collected photographs and bankfull width measurements both upstream and downstream of the crossing, but only collected pebble counts upstream of the crossing. Pebble count data for the reach downstream of the crossing was collected by GeoEngineers as part of their site visit to Crossing 991999.

General Site Description

Crossing 991572 is located along SR 307 at mile marker 01.45 in Kitsap County, Washington. This crossing carries run-off from the nearby farm fields and low-density residential areas east and northeast of Poulsbo through an unnamed tributary (UNT) to Dogfish Creek. The WDFW Level A Culvert Assessment report, conducted in July 1999, states that the crossing is a 4-foot-diameter round corrugated steel (CST) culvert with a length of 33.80 m (110.9 ft) and a slope of 2.15%. The slope of the crossing was not verified during the site visit, however, the culvert size was measured to be 4 feet.

The inlet opening is at the toe of the road fill with steep banks (approximately 50% slope). The inlet is covered by several 6- to 10-inch logs. The outlet has no wingwalls or apron. Immediately downstream of the culvert outlet is a scour pool at the confluence with another unnamed tributary flowing from north to south. The stream turns approximately 90 degrees to flow west along the road embankment.

Bankfull Width:

Bankfull width (BFW) measurements were taken in a total of six locations. Three bankfull width measurements were taken upstream of the crossing by PACE on December 2, and three were taken downstream of the crossing by GeoEngineers on December 10.

Table 1 Bankfull Width Location and Measurements

	Approximate Distance from Crossing (ft)	Bankfull Width (ft)	Source/Date
Upstream	100	17.0	PACE (December 2021)
	165	8.4	PACE (December 2021)
	200	9.9	PACE (December 2021)
	Upstream Average	**11.8**	
Downstream	240	9.0	PACE (December 2021)
	635	11.0	PACE (December 2021)
	870	10.6	GeoEngineers (December 2021)
	950	11.9	GeoEngineers (December 2021)
	1060	10.2	GeoEngineers (December 2021)
	Downstream Average	**10.5**	

Reference Reach:

The reference reach is approximately between 850 and 1,050 feet downstream of Site ID 991572. The reference reach for Site ID 991572 is the same reference reach as for Site ID 991999, NE Dogfish Ck., which is approximately 1,150 feet downstream of the outlet of 991572. The two crossings are approximately 1,200 feet apart from each other but share similar characteristics. The reference reach includes flow from an unnamed tributary which joins the UNT to Dogfish Creek just downstream of the Project Crossing. The slope of the reference reach is the same as the reach downstream of the Project Crossing (Figure 3).

Immediately downstream of the crossing the site is incised and confined against the road prism of SR 307. Approximately 180 feet downstream of the crossing the channel bends away from the SR 307. The channel from 200 feet downstream until 1050 feet downstream is representative of natural channel conditions. From 200 feet downstream until the next crossing downstream the channel is uniform in slope and geometry.

The crossing is located at a transition point in valley width. The channel upstream of the crossing has wider floodplain benches and appears to be less confined than the channel downstream of the crossing. The wider valley upstream is associated with an alluvial fan at the confluence of tributaries and the valley becomes more confined downstream of the crossing.

Five bankfull widths were measured downstream of the crossing. All bankfull width measurements were between 9 and 11.9 feet for an average of 10.5 feet. Upstream of the crossing three bankfull widths were measured. Two bankfull widths were approximately in line with the downstream measurements, but one measurement was 17 feet. This measurement was taken where the bank had a very shallow slope up to a wide floodplain bench. The channel morphology at this location was a wide shallow riffle.

Data Collection:

The entire attendance list participated in the collection of data, with the date next to their name being when they participated. Data was collected upstream on December 2, from the inlet until roughly 250 feet upstream of the inlet. On December 9, photographs were collected downstream until about 700 feet from the outlet. Additionally, GeoEngineers collected photographs, bankfull width measurements, and pebble counts as part of a site visit for Crossing 991999 which is approximately 1,150 feet downstream of the outlet of 991572. GeoEngineers collected data in a reach which is approximately 900 to 1,000 feet downstream of 991572. Data collected included:

- General site observations
- Bankfull width measurements
- Other channel geometry measurements (bank height, channel width, water depth, etc.)
- Pebble counts

Observations:

Site conditions

Crossing 991572 is a 4-foot corrugated metal pipe (CMP). At the upstream end it is heavily covered by woody debris (Photo 1). Some of the woody debris appeared to have originated on the banks near the crossing, but some of the debris appeared to have been transported to this site from upstream. The culvert was relatively undamaged at the upstream end. There were several angular boulders near the culvert inlet, which likely were placed during construction or maintenance activities.

The outlet of Site ID 991572 was free of debris and undamaged. There was a small glide which was approximately 1 foot deep immediately downstream of the outlet. This pool extended for approximately 6 feet downstream. The substrate of this pool was silt, sand, and several 10-inch cobbles. It was unclear if the larger substrate was placed there or transported from upstream. Approximately 15 feet downstream of the outlet is a confluence with an unnamed tributary flowing from the north.

Geomorphology

Upstream conditions

Immediately upstream of the inlet of the crossing is a long shallow pool (Photo 1). On either side of the culvert are large cobbles or small boulders which are angular and were likely placed as part of construction or maintenance. The pool continues upstream for approximately 40 feet (Photo 3). The left bank is undercut and the thalweg is on that side of the channel. The right bank is approximately 2 feet tall and connects to a bench, while the left bank is the valley wall and had no bench. The right bench was very densely covered in riparian vegetation and was not accessed. The channel shifts away from the valley wall and there are overbank benches on both the right and left banks approximately 90 feet upstream (Photo 4). Approximately 70 to 100 feet upstream was a shallow riffle which was composed of sands and gravels (Photo 5). Several pieces of large wood were in the channel at the upstream end of the riffle.

Upstream of the riffle was a 2-foot-deep scour pool. The pool appeared to have been scoured by flow being forced under the large wood (Photo 6). The banks on both the right and left side here were about 2 feet tall before opening up onto high flow benches. The banks were highly vegetated, sloped at approximately a 2V:1H slope and appeared stable. Upstream of the pool was a 50-foot long riffle with a log lying along the toe of the right bank (Photo 7). In this reach both banks had a gentle slope, were densely vegetated, and opened up to a high flow bench approximately 2 feet above the thalweg. The substrate was predominately sands and gravels, but there were isolated boulders which were approximately 1.2 feet across (Photo 9).

Approximately 220 feet upstream the channel takes a 90-degree turn, and there is a shallow scour pool along the outer edge of the bend (Photo 10). There is a channel-spanning log, but it does not seem to be interacting with the flows. Fifty feet upstream of the 90-degree bend is a confluence where UNT to Dogfish Creek meets another unnamed tributary which flow from the south (Photo 11). The channel is wider at this tributary junction, and there are several shallow gravel bars, indicating this area is depositional. The banks were approximately 1 foot tall in this area and it appeared that during high flows water was carried in shallow overbank channels.

Downstream conditions

The outlet of the crossing is a 1-foot-deep glide, which extends for approximately 6 feet. Downstream of the glide there is a large pool at the confluence of UNT to Dogfish Creek and another unnamed tributary which flows from the north (Photo 12). This pool is approximately 15 feet across and is 2 to 3 feet deep. The outlet of the pool is flowing southwest, so UNT to Dogfish creek makes a 90-degree turn at the pool. The substrate in this area is sand, silt, and cobbles, with occasional small boulders. There was large asphalt rubble debris observed on the bed 180 feet downstream of the outlet. The tributary from the north has a crossing immediately upstream of the pool (Site ID 930880) which was a fish passage barrier but was replaced in 2013. The confluence pool has several mature cedars on the bank which are being undercut and have exposed roots. A short riffle with gravels and cobbles is controlling the outlet of the pool.

Downstream of the confluence pool the channel flows along the embankment of SR 307 for approximately 170 feet (Photo 13). The channel substrate is sands, gravels, cobbles, and boulders in this reach. The left bank, the SR 307 embankment, is a continuous slope down to the channel, without any floodplain. The right bank is approximately 3 to

5 feet tall and is undercut by 1 to 2 feet along much of this reach (Photo 14). There were no indications that flow spilled onto the floodplain bench on the right side. It is unclear when the stream incised and disconnected from the floodplains, but given the size of the conifers in the floodplain, the stream has not regularly inundated the floodplain in decades.

Approximately 180 feet downstream the stream turns north to flow away from SR 307. There is a debris jam and a scour pool at this bend (Photo 15). The scour pool is approximately 2 feet deep. As the stream flows away from the road, the slope of the left bank becomes gentler and the floodplain terrace on the left lowers to approximately 2 feet above the channel. Approximately 330 feet downstream the channel bends again back to the left to flow southwest, and at the bend there is another fallen tree with a rootwad creating a 1.3-foot-deep scour pool (Photo 17).

Aquatic Habitat Type and Location

<u>Upstream conditions</u>

Upstream of the Site ID 991572 has some high-quality rearing and spawning habitat, although much of the reach is a shallow riffle. The substrate size in the riffles appears too fine for spawning, although some cleaning of the fines during redd construction may result in a suitable substrate size distribution (Photo 7, Photo 9). At 120 and 175 feet upstream wood has forced flow and created scour pools which can be used as rearing habitat for anadromous species (Photo 6, Photo 9). There are undercut banks along the left bank near the culvert inlet offering predator refuge (Photo 3). There is vegetation lining the banks for the entire surveyed reach which is overhanging the channel and adding woody debris to the channel, which increases the invertebrate populations and feeding opportunities for salmonids (Photo 4).

<u>Downstream conditions</u>

Downstream of Site ID 991572 UNT to Dogfish Creek has high quality habitat. There is abundant large woody material and boulders creating diverse habitat. Immediately downstream of the crossing, a 2-foot-deep pool at the confluence with a tributary from the north offers rearing habitat for salmonids of all life stages. There is a deep undercut bank along the edge of the pool which offers cover habitat (Photo 12). The outlet of the pool is a riffle with abundant gravels and cobbles and likely could support spawning of anadromous and resident fish (Photo 13). From approximately 50 to 150 feet downstream the channel runs along the road embankment and has few pools. However, coarse substrate and habitat boulders create small pocket pools which create rearing habitat for juveniles and resident fish (Photo 14). The undercut banks also create predator refuge in this reach. Farther downstream there continues to be high quality rearing habitat in scour pools created by large wood every 50 to 100 feet, with riffle or glide morphologies offering spawning opportunities between the pools (Photo 15, Photo 17).

Large Woody Material Location and Quantity

<u>Upstream conditions</u>

There is abundant LWM in the channel and on the banks upstream of the crossing. The crossing inlet is covered by several pieces of LWM, some of which appear to have fallen from the banks near the crossing and some of which appear to have racked on the culvert during high flows (Photo 1). In the reach 0 to 50 feet upstream of the inlet there are several pieces of LWM on the banks or extending out to the toe of the bank, but there the LWM has not forced any large scour pools (Photo 3). There is a large scour pool approximately 180 feet upstream of the crossing, where a large spanning log forces water underneath it (Photo 5, Photo 6). Approximately 130 feet upstream of the crossing there is a 50-foot log lying along the toe of the bank, with another log spanning the channel and lying on top of the toe log (Photo 7). There are two more spanning logs at 175 feet upstream of the crossing (Photo 9) and 195 feet upstream (Photo 10). At the confluence approximately 260 feet upstream there are several more pieces of the LWM (Photo 11).

<u>Downstream conditions</u>

Immediately downstream of the crossing until the first debris jam at 180-feet downstream there is some woody material which is less than 1-foot diameter (Photo 14). There are also exposed roots on the undercut banks. In this reach there are many pieces of woody material lying on the banks. At approximately 180-feet downstream of the crossing is a 2-foot-diameter spanning log which has racked several other 1-to-2-foot logs and other small organic debris (Photo 15). There is abundant 1- to 3-foot wood which is spanning the channel approximately 270 feet downstream (Photo 16). Some of this appeared to have fallen fairly recently while other logs were moss covered and clearly had not moved in years. At 330 feet downstream there is a 1.5-foot log with a rootwad creating a scour pool (Photo 17).

Vegetation

Upstream conditions

In the upstream reach, vegetation in the riparian corridor is composed of salmonberry (*Rubus spectabilis*), sword fern (*Polystichum munitum*), spreading wood fern (*Dryopteris expansa*), western red cedar (*Thuja plicata*) and large leaf maple (*Acer macrophyllum*). There are occasional evergreens such as western red cedar on the channel banks, but they are predominantly higher up on the slopes.

Downstream conditions

The downstream reach has a similar plant assemblage, but the banks have a higher degree of evergreen trees. The banks and floodplains closer to the channel are dominated by salmonberry, sword fern, and other riparian vegetation. The overbank bench is dominated by cedars and other evergreens.

Pebble Counts:

Two Wolman pebble counts were collected by PACE and three Wolman pebble counts were collected by GeoEngineers. PACE collected two pebble counts upstream of the crossing, approximately 100 and 200 feet upstream. GeoEngineers collected three pebble counts in the reference reach, which were 800, 690, and 600 feet downstream of 991572. These were collected as part of the field data collection for Site ID 991999. The channel substrate consisted predominately of coarse gravel, sand, and some small cobbles with the occasional 1- to 3-man boulder found in the channel. See Figure 2 for an overview map showing the locations of the pebble counts.

Table 2. Substrate Distribution

Pebble Count	Downstream (reference reach)			Upstream	
Diameter Percentile	PC3 (in)	PC4 (in)	PC5 (in)	PC1 (in)	PC2 (in)
D_{100}	5.0	3.5	3.5	3.5	10.1
D_{84}	1.8	1.7	1.9	1.5	1.7
D_{50}	1.0	0.8	1.0	0.7	0.5
D_{16}	0.5	0.4	0.4	0.1	0.2

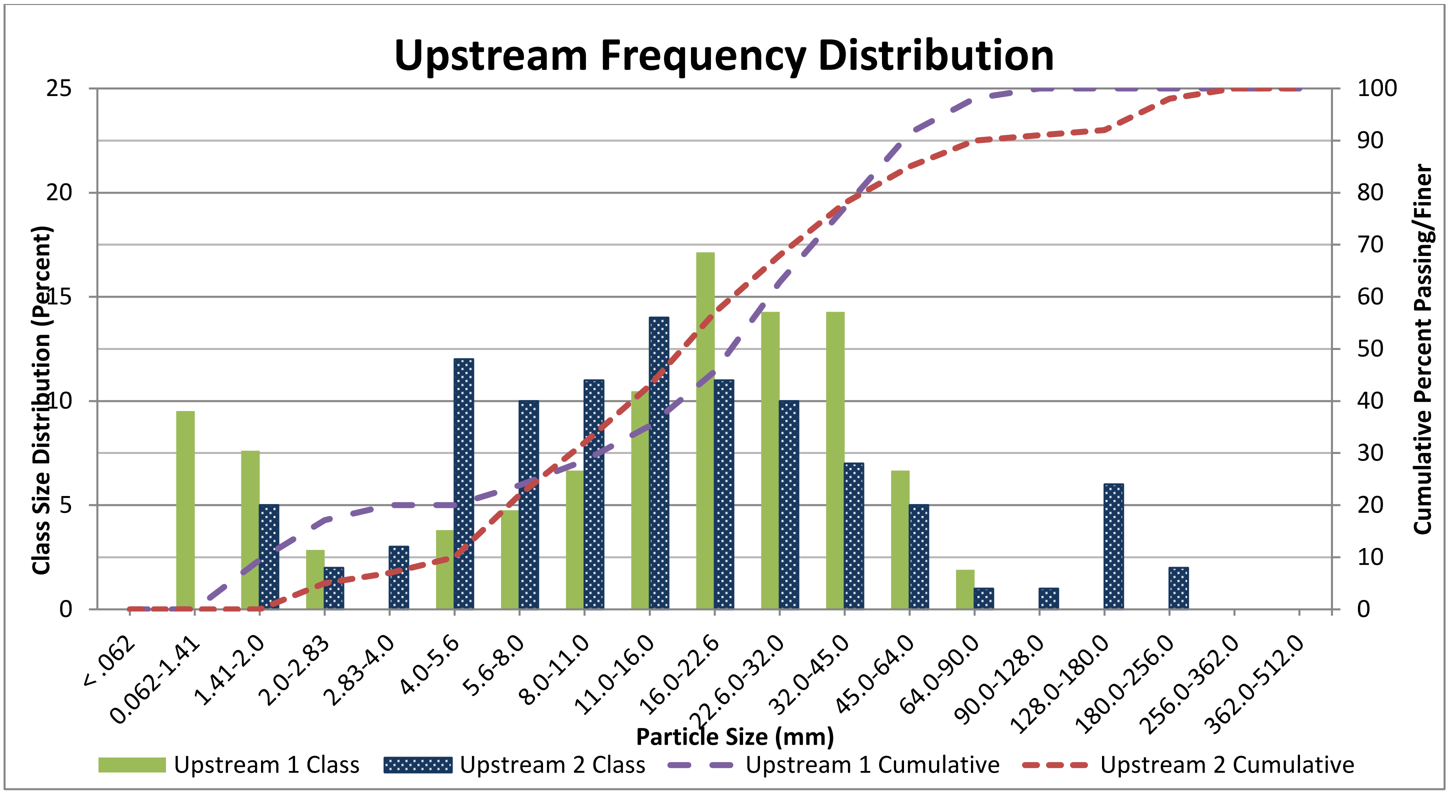


Figure 1. Upstream Pebble Count Gradation

Photos:

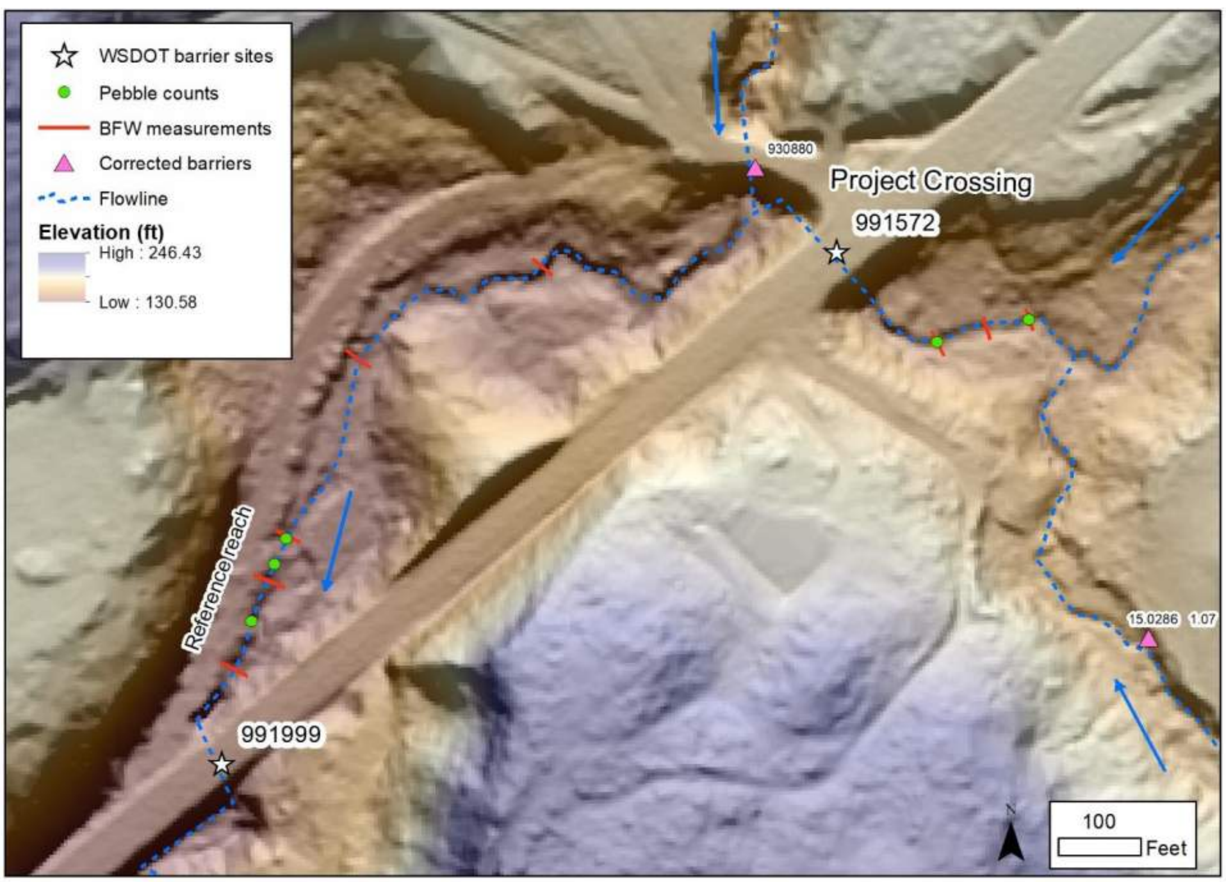


Figure 2. Overview of Project Crossing 991572 and the new crossing downstream, Site ID 991999

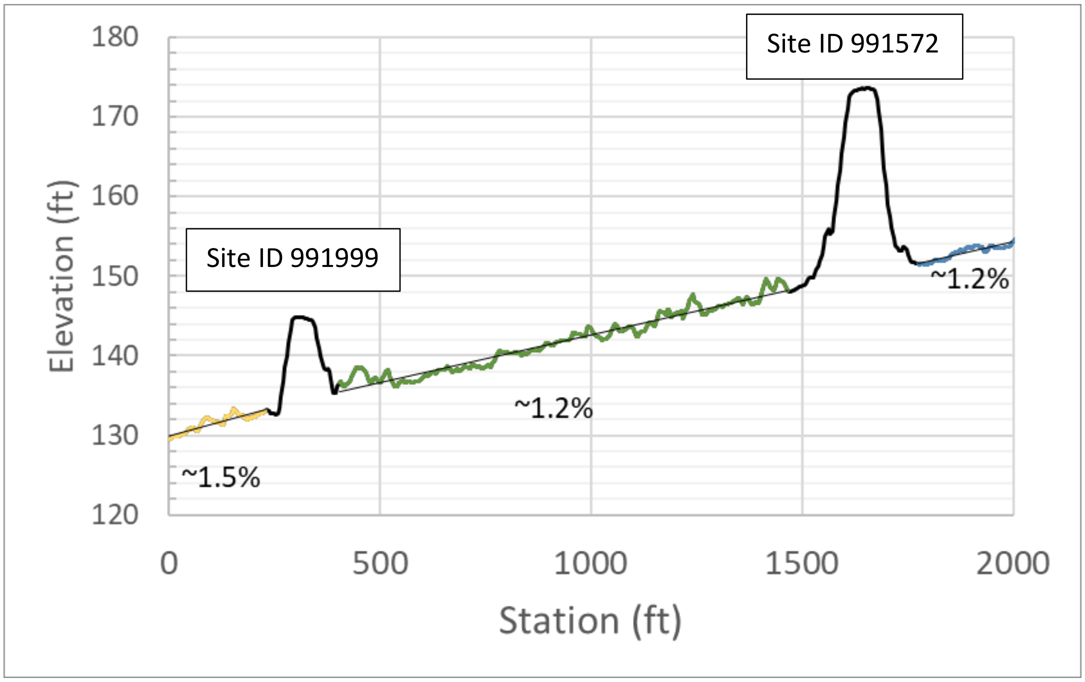


Figure 3. Longitudinal profile from LiDAR

Photo 1. Inlet of Site ID 991572 covered by woody debris

Photo 2. Outlet of Site ID 991572

Photo 3. Approximately 50 feet upstream of the inlet of Site ID 991572, looking downstream


Photo 4. Approximately 90 feet upstream of the crossing, looking upstream

Photo 5. Approximately 100 feet upstream of the crossing, looking upstream

Photo 6. Approximately 110 feet upstream, looking upstream

Photo 7. Approximately 130 feet upstream of the crossing, looking upstream

Photo 8. Approximately 160 feet upstream, looking upstream

Photo 9. Channel substrate and spanning log approximately 175 feet upstream, looking upstream

Photo 10. Approximately 195 feet upstream, looking upstream

Photo 11. Confluence of UNT to Dogfish Creek with another tributary flowing from the south, looking upstream

Photo 12. Confluence pool just downstream of the Project Crossing (991572) and the corrected barrier (930880)

Photo 13. Channel approximately 50 to 100 feet downstream of the crossing


Photo 14. Approximately 80 feet downstream of the crossing, tall right bank with deep undercut


Photo 15. Debris jam and scour pool at bend in stream approximately 180 feet downstream

Photo 16. Woody material and gravel substrate approximately 270 feet downstream.

Photo 17. Scour pool at 90-degree bend approximately 330 feet downstream.

Samples:				
Work within the wetted perimeter may only occur during the time periods authorized in the APP ID 21036 entitled "Allowable Freshwater Work Times May 2018". Work outside of the wetted perimeter may occur year-round. APPS website: https://www.govonlinesaas.com/WA/WDFW/Public/Client/WA_WDFW/Shared/Pages/Main/Login.aspx				
Were any sample(s) collected from below the OHWM?	*No ☐ If no, then stop here.*			
	Yes ☐ If yes, then fill out the proceeding section for each sample.			

Sample #:	Work Start:	Work End:	Latitude:	Longitude:

Summary/description of location:
Summarize/describe the sample location.
Description of work below the OHWL:
Describe the work below the OHWL, including equipment used and quantity of sediment sampled.
Description of problems encountered:
Describe any problems encountered, such as provision violations, notification, corrective action, and impacts to fish life and water quality from problems that arose.

Concurrence Meeting

Concurrence Meeting	Date: Feb 2nd, 2022	Time of Arrival: 2:30 pm
Prepared By: C Nicol, S Sheldon	Weather: Partly cloudy	Time of Departure: 4:30 pm

Attendance List:

Name	Organization	Role
Kate Fauver	WSDOT	Transportation Planner
Alison O'Sullivan	Suquamish Tribe	Biologist
Damon Romero	WSDOT	Biologist
Dave Molenaar	WSDOT	Habitat Biologist
Heather Pittman	WSDOT	State Hydraulic Engineer
Amber Martens	WDFW	Biologist
Shawn Stanley	WDFW	Habitat Engineer
Marla Powers	Port Gamble S'Klallam Tribe	Environmental Planner
Hunter Henderson	WSDOT	Transportation Specialist
Colin Nicol	PACE	Environmental Scientist
Shane Sheldon	PACE	Engineer

Bankfull Width:

Summarize on-site discussion, describe measurements, and concurrence or decisions made that help to inform the design.

- Upstream of the crossing the concurrence group measured: 9.0 ft, 11.0 ft, 10.0 ft, 11.0 ft.
- Downstream of the crossing the concurrence group measured: 14 ft, 10 ft, 13.5 ft, 12 ft.
- Group looked at spot of 17.0 ft measurement from SV#2 and agreed it should not be used in average. Scalloped bank and not representative of the overall site.
- On the upstream side all bankfull width measurements are outside of any potential backwater.
- Averaging in bankfull widths from the reference reach will add a factor of safety because there is a larger drainage area to the reference reach than to the crossing inlet. Group agreed this was a good approach
- [Added note Feb 24, 2022] – Email from Bill Bumback clarifying that a design average of 12.4 feet as per Amber Martin's notes should be used
    - "After discussing Amber's comments with WSDOT (Kate Fauver, Cade Roler, Heather Pittman, Dennis Engel, Damon Romero, and Nazmul Alam) this morning, the consensus is that we should update the tracker with the widths Amber noted and make note in the PHDs that these are the widths arrived at through the concurrence process. "

Table 3. All bankfull widths measured and concurrence average

	Approximate Distance from Crossing (ft)	Bankfull Width (ft)	Used in average	Source/Date
Upstream	100	17.0	N	PACE (December 2021)
	165	8.4	N	PACE (December 2021)
	200	9.9	N	PACE (December 2021)
	165	9	N	Concurrence site visit (February 2022)
	185	11	N	Concurrence site visit (February 2022)
	200	10	N	Concurrence site visit (February 2022)
	220	11	N	Concurrence site visit (February 2022)
Downstream	240	9.0	N	PACE (December 2021)
	635	11.0	N	PACE (December 2021)
	870	10.6	N	GeoEngineers (December 2021)
	950	11.9	N	GeoEngineers (December 2021)
	1060	10.2	N	GeoEngineers (December 2021)
	600	14	Y	Concurrence site visit (February 2022)

	615	10	Y	Concurrence site visit (February 2022)
	655	13.5	Y	Concurrence site visit (February 2022)
	685	12	Y	Concurrence site visit (February 2022)
	Design Average	**12.4**		

Reference Reach:

Summarize on site discussion, concurrence and/or appropriateness of selected reference reach.

- Group agrees road is confining stream just downstream of crossing and using the same reference reach as Site ID 991999 makes sense.
- This also allows for some continuity in the design between 991572 and 991999.
- Averaging in bankfull widths from the reference reach will add a factor of safety because there is a larger drainage area to the reference reach than to the crossing inlet.

Observations:

Summarize on site discussions, any perceived/known project constraints, or other details that help to inform the design.

- Alison noted the creek is locally known as Northeast Dogfish Creek.
- Site ID 990123 might be a good example crossing to look at on Dogfish Creek.
- Site upstream is “unconfined” in the sense that the planform may be susceptible to change given forcing features such as wood. No old evergreens on banks.
- Possible maintenance issue where stream runs along SR 307 just downstream of crossing.
    - Realignment to move stream away from road should be considered.
- This is possibly a wildlife crossing. Is there a memo?

Photos:

Any relevant photographs placed here with descriptions.

Photo 18. Concurrence group taking a bankfull width measurement in the upstream end of the reference reach

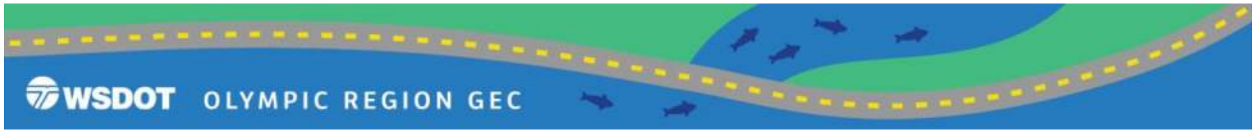
Photo 19. Concurrence group taking a bankfull width measurement in the reference reach

Fish Passage Project Site Visit - Determining Project Complexity

PROJECT NAME:	Unnamed to Dogfish Creek -- Fish Passage Barrier Removal
WDFW SITE ID:	991572
STATE ROUTE/MILEPOST:	SR 307 MP 1.45
SITE VISIT DATE:	December 2nd (upstream) and December 9th (downstream)
ATTENDEES:	Shane Sheldon, Colin Nicol, Tasha Wang, Miranda Smith, Henry Moen
ANTICIPATED LEVEL OF PROJECT COMPLEXITY - Low/Medium/High (*additional considerations or red flags may trigger the need for new discussions*):	Medium. The site appears to be unconfined on the upstream side and confined on the downstream side. Private driveways on both sides of the stream may make it difficult to significantly widen the active floodplain.
IN WATER WORK WINDOW	August 1 – August 15

The following elements of projects should be discussed before the production of a Preliminary Hydraulic Design by members of WSDOT and WDFW to identify the level of complexity for each site, and corresponding communication and review. While certain elements may be categorized as indicators of a low/medium/high complexity project, these are only suggestions, and newly acquired information may change the level of complexity during a project. The ultimate documentation category for a given site is up to both WSDOT and WDFW, considering both site characteristics and synergistic effects.

Discuss the following elements as they apply to the project. Rank each element as low, medium, or high in complexity. If there are items that need follow-up, mark those and provide a brief description in the column labeled, "Is follow up needed on this item?" The assigned level of complexity determines the appropriate agreed upon review from WDFW (see review parameters here (*final full doc goes here*)). Ultimately, WSDOT needs to acquire an HPA from WDFW for fish passage projects and the agreed upon communication and review of project elements will contribute to efficiencies in the permitting process.



Fish Passage Project Site Visit - Determining Project Complexity

Project Elements (anticipated)	Low Complexity	Medium Complexity	High Complexity	Is follow up needed on this item?
Stream grading	X			
Risk of degradation/aggradation	X			No signs of recent aggradation or degradation, but the downstream end does appear to have incised into the historic floodplain creating a wide terrace
Channel realignment		X		It is likely the stream was moved to the left valley wall at the inlet. However, the stream can't be moved back to the middle of the valley because on the downstream end a private road confines the valley
Expected stream movement		X		Upstream of the crossing the channel appears to be unconfined and has a floodplain bench, while downstream the channel is more incised and confined within the banks
Gradient	X			
Potential for backwater impacts	X			
Meeting requirements for freeboard	X			
Stream size, and Bankfull Width	X			
Slope ratio	X			
Sediment supply	X			
Meeting stream simulation	X			
Channel confinement		X		Unconfined upstream, confined downstream
Geotech or seismic considerations	X			
Tidal influence	X			
Alluvial fan	X			Small alluvial fan at confluence 250 feet upstream of the crossing. Assuming we tie in well below the confluence it should not pose any issues
Fill depth above barrier	X			
Presence of other nearby barriers		X		Immediately downstream is a confluence with a corrected barrier 930880. The design regrade will possibly impact the outlet of that crossing. No indications that crossing is unstable
Presence of nearby infrastructure		X		Private road and crossing 930880 immediately downstream
Need for bank protection	X			

Fish Passage Project Site Visit - Determining Project Complexity

Floodplain utilization ratio		X		Unconfined upstream, confined downstream
Other:				

Summary - Streambed Material Design (Critical Shear)

Project:	NE Dogfish Creek 991572
By:	Colin Nicol

Streambed Design Gradation: Critical Shear				
	D_{100}	D_{84}	D_{50}	D_{16}
ft	0.50	0.20	0.08	0.01
in	6.0	2.4	0.9	0.1
mm	152	60	23.9	1.6

(blue)	Input by user
(yellow)	Calculated by sheet

Streambed Mobility/Stability Analysis

Modified Shields Approach

References:

Stream Simulation: An Ecological Approach to Providing Passage for Aquatic Organizms at Road-Stream Crossings

Appendix E--Methods for Streambed Mobility/Stability Analysis

Limitations:	Project Applicable
D_{84} must be between 0.40 in and 10 in	Yes
uniform bed material (Di < 20-30 times D50)	Yes
Slopes less than 5%	Yes
Sand/gravel streams with high relative submergence	Yes

γ_s	165	specific weight of sediment particle (lb/ft^3)
γ	62.4	specific weight of water (1b/ft^3)
τ_{D50}	0.047	Dimensionless Shields parameter for D50, use table E.1 of USFS manual or assume 0.045 for poorly sorted channel bed

* All the shear stress were taken within the crossing, not within the regraded channel.

Flow	2-YR Max.	2-YR Ave.	100-YR Max.	100--YR Ave.
Average Modeled Shear Stress (lb/ft^2)	1.54	1.10	2.27	2.14
τ_{ci}		Structure		
1.13	Motion	No Motion	Motion	Motion
1.09	Motion	Motion	Motion	Motion
1.05	Motion	Motion	Motion	Motion
0.99	Motion	Motion	Motion	Motion
0.92	Motion	Motion	Motion	Motion
0.87	Motion	Motion	Motion	Motion
0.81	Motion	Motion	Motion	Motion
0.77	Motion	Motion	Motion	Motion
0.72	Motion	Motion	Motion	Motion
0.66	Motion	Motion	Motion	Motion
0.62	Motion	Motion	Motion	Motion
0.58	Motion	Motion	Motion	Motion
0.54	Motion	Motion	Motion	Motion
0.51	Motion	Motion	Motion	Motion
0.47	Motion	Motion	Motion	Motion
0.44	Motion	Motion	Motion	Motion
0.39	Motion	Motion	Motion	Motion
0.35	Motion	Motion	Motion	Motion

Mixture D50 (ft)		in	mm
	D16	0.06	1.64
0.079	D50	0.94	23.95
	D84	2.37	60.22
	D95	5.00	127.00
	D100	6.00	152.40

Determining Aggregate Proportions

Per WSDOT Standard Specifications 9-03.11

	Rock Size [in]	Rock Size [mm]	Streambed Sediment	Streambed Cobbles 4"	6"	8"	10"	12"	Streambed Boulders 12"-18"	18"-28"	28"-36"	D_{size}
	36.0	914									100	100.0
	32.0	813									50	100.0
	28.0	711								100		100.0
	23.0	584								50		100.0
	18.0	457							100			100.0
	15.0	381							50			100.0
	12.0	305						100				100.0
	10.0	254					100	80				100.0
	8.0	203				100	80	68				100.0
	6.0	152			100	80	68	57				100.0
	5.0	127			80	68	57	45				95.0
4	4.0	102		100	71	57	45	39				92.8
3	3.0	76.2		80	63	45	38	34				90.6
2.5	2.5	63.5	100	65	54	37	32	28				88.4
2	2.0	51	80	50	45	29	25	22				71.3
1.5	1.5	38.1	73	35	32	21	18	16				62.3
1	1.0	25.4	65	20	18	13	12	11				53.3
0.8	0.75	19.1	50	5	5	5	5	5				38.8
0.2	No. 4 =	4.75	35									26.3
0	No. 40 =	0.425	16									12.0
0	No. 200 =	0.0750	7									5.3
	% per category		75	0	25	0	0	0	0	0	0	--> 100%
	% Cobble & Sediment		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	100.0%

Proposed Streambed Gradation	in	mm	Average Pebble Count	in	mm	% diff
D16	0.06	1.64	D16	0.30	7.62	
D50	0.94	23.95	D50	0.80	20.32	18%
D84	2.37	60.22	D84	1.70	43.18	
D95	5.00	127.00	D95	3.10	78.74	
D100	6.00	152.40	D100	5.10	129.54	

Summary - Meander Bar Tail Material Design (Critical Shear)

Project:	NE Dogfish Creek 991572
By:	Colin Nicol

Streambed Design Gradation: Critical Shear				
	D_{100}	D_{84}	D_{50}	D_{16}
ft	0.67	0.41	0.14	0.01
in	8.0	5.0	1.7	0.2
mm	203	126	43.4	4.1

(blue)	Input by user
(yellow)	Calculated by sheet

Streambed Mobility/Stability Analysis

Modified Shields Approach

References:

Stream Simulation: An Ecological Approach to Providing Passage for Aquatic Organizms at Road-Stream Crossings

Appendix E--Methods for Streambed Mobility/Stability Analysis

Limitations:	Project Applicable
D_{84} must be between 0.40 in and 10 in	Yes
uniform bed material (Di < 20-30 times D50)	Yes
Slopes less than 5%	Yes
Sand/gravel streams with high relative submergence	Yes

γ_s	165	specific weight of sediment particle (lb/ft^3)
γ	62.4	specific weight of water (1b/ft^3)
τ_{D50}	0.047	Dimensionless Shields parameter for D50, use table E.1 of USFS manual or assume 0.045 for poorly sorted channel bed

* All the shear stress were taken within the crossing, not within the regraded channel.

Flow	2-YR Max.	2-YR Ave.	100-YR Max.	100--YR Ave.
Average Modeled Shear Stress (lb/ft^2)	1.54	1.10	2.27	2.14
τ_{ci}		Structure		
1.71	No Motion	No Motion	Motion	Motion
1.65	No Motion	No Motion	Motion	Motion
1.59	No Motion	No Motion	Motion	Motion
1.50	Motion	No Motion	Motion	Motion
1.39	Motion	No Motion	Motion	Motion
1.32	Motion	No Motion	Motion	Motion
1.23	Motion	No Motion	Motion	Motion
1.17	Motion	No Motion	Motion	Motion
1.09	Motion	Motion	Motion	Motion
1.00	Motion	Motion	Motion	Motion
0.95	Motion	Motion	Motion	Motion
0.89	Motion	Motion	Motion	Motion
0.81	Motion	Motion	Motion	Motion
0.77	Motion	Motion	Motion	Motion
0.72	Motion	Motion	Motion	Motion
0.66	Motion	Motion	Motion	Motion
0.58	Motion	Motion	Motion	Motion
0.54	Motion	Motion	Motion	Motion

Mixture D50 (ft)		in	mm
	D16	0.16	4.07
0.142	D50	1.71	43.43
	D84	4.97	126.27
	D95	7.00	177.80
	D100	8.00	203.20

Determining Aggregate Proportions

Per WSDOT Standard Specifications 9-03.11

	Rock Size [in]	[mm]	Streambed Sediment	Streambed Cobbles 4"	6"	8"	10"	12"	Streambed Boulders 12"-18"	18"-28"	28"-36"	D_{size}
	36.0	914									100	100.0
	32.0	813									50	100.0
	28.0	711								100		100.0
	23.0	584								50		100.0
	18.0	457							100			100.0
	15.0	381							50			100.0
	12.0	305						100				100.0
	10.0	254					100	80				100.0
	8.0	203				100	80	68				100.0
	6.0	152			100	80	68	57				90.0
	5.0	127			80	68	57	45				84.2
4	4.0	102		100	71	57	45	39				78.3
3	3.0	76.2		80	63	45	38	34				72.5
2.5	2.5	63.5	100	65	54	37	32	28				68.5
2	2.0	51	80	50	45	29	25	22				54.5
1.5	1.5	38.1	73	35	32	21	18	16				46.8
1	1.0	25.4	65	20	18	13	12	11				39.0
0.8	0.75	19.1	50	5	5	5	5	5				27.5
0.2	No. 4 =	4.75	35									17.5
0	No. 40 =	0.425	16									8.0
0	No. 200 =	0.0750	7									3.5
	% per category		50	0	0	50	0	0	0	0	0	--> 100%
	% Cobble & Sediment		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	100.0%

Proposed Meander Bar Tail Gradation	in	mm	Average Pebble Count	in	mm
D16	0.2	4.07	D16	0.30	7.62
D50	1.7	43.43	D50	0.80	20.32
D84	5.0	126.27	D84	1.70	43.18
D95	7.0	177.80	D95	3.10	78.74
D100	8.0	203.20	D100	5.10	129.54

Stream Name NE Dogfish Creek 991572

Meander bar head sizing

g	ρw	ρs	τ*c
9.81	1000	2650	0.044

	inch	feet	
D50 (ft)	5.3	0.4	meander bar tail D50

CRITICAL SHEAR

	τ lb/ft2	τ	D (m)	D (ft)	D (in)
Q2 Ave	1.2	57.5	0.1	0.3	3.2
Q100 Ave	2.0	95.8	0.1	0.4	5.3
Q2max	1.6	76.6	0.1	0.4	4.2
Q100max	3.9	186.7	0.3	0.9	10.3

$$D = \frac{\tau}{\tau_c^* * (\rho s - \rho w) * g}$$

Modified Critical Shear

	τ ci (lb/ft2)	Di (ft)	Di (in)
Q2 Ave	1.2	0.1	1.0
Q100 Ave	2.0	0.4	5.4
Q2max	1.6	0.2	2.5
Q100max	3.9	4.1	49.6
Qtest	2.9	1.5	18.5

$$D_i = \left(\frac{\tau_{ci}}{102.6 * \tau_{D50} * D_{50}{}^{0.7}}\right)^{\left(\frac{1}{0.3}\right)}$$

$$V_{min} = 0.86 \{2 g [(SG_s - SG_w)/SG_w]\}^{0.5} D^{0.5} \qquad \text{Isbash, 1936}$$

where: V_{min} = minimum velocity
g = gravity = 32.2 ft/s^2 = 9.81 m/s^2
SG_s = specific gravity of stone, varies with the type of stone—generally ranges from 2.2 to 3.2
SG_w = specific gravity of water, generally assumed = 1.0
D = diameter of the stone (assuming a spherical shape)

Rearranged to solve for the minimum diameter of stone (D_{min}) necessary to withstand a given design velocity (V), Isbash's equation becomes:

$$\mathbf{D_{min} = V^2/\{1.479\, g\, [(SG_s - SG_w)/SG_w]\}}$$

Costa studied nine steep bedrock channels in the Colorado Front Range to test the accuracy of velocity and depth estimates for historical peak floods based on the size of boulders transported during the flood event. He developed the following equation by taking the arithmetic average of four commonly used methods for computing stream velocity.

$$V_{avg} = 9.571\, D^{0.487} \qquad \text{Costa, 1983}$$

where: V_{avg} = average velocity (ft/s)
D = diameter of the stone (ft)

Rearranging to solve for D:

$$\mathbf{D_{min} = (V_{avg}/9.571)^{2.05}}$$

Equation E.5 $\tau_{ci} = \tau_{D50} (\gamma_s - \gamma) D_i^{0.3} D_{50}^{0.7}$

where:

τ_{ci} is the critical shear stress at which the sediment particle of interest begins to move (lb/ft^2 or N/m^2).

τ_{D50} is the dimensionless Shields parameter for D_{50} particle size (this value can either be obtained from table E.1, or the value 0.045 can be used for a poorly sorted channel bed).

D_{50} is the diameter (ft or m) of the median or 50th percentile particle size of the channel bed .

D_i is the diameter (ft or m) of the particle size of interest. For stream simulation the particle size of interest is typically D_{84} and/or D_{95}.

Assuming γ_s = 165 lb/ft^3 and γ = 62.4 lb/ft^3, equation 5 can be simplified to:

Equation E.6 $\tau_{ci} = 102.6\, \tau_{D50} D_i^{0.3} D_{50}^{0.7}$

g	32.2 ft/s^2
SG_s	2.65
SG_w	1

Recurrence	Velocity (max in crossing)	Isbash Dmin		Costa Dmin	
Q2	3.7 ft/s	0.2 ft	2.1 in	0.1 ft	1.7 in
Q100	5.5 ft/s	0.4 ft	4.6 in	0.3 ft	3.9 in
Q2080 - 100	6.1 ft/s	0.5 ft	5.7 in	0.4 ft	4.8 in
Q500	5.5 ft/s	0.4 ft	4.6 in	0.3 ft	3.9 in

Table 1 Limiting Shear Stress and Velocity For Uniform Noncohesive Sediments

Class name	d_s (in)	φ (deg)	τ_c	τ_{cr} (lb/sf)	V_{*c} (ft/s)
Boulder					
Very large	>80	42	0.054	37.4	4.36
Large	>40	42	0.054	18.7	3.08
Medium	>20	42	0.054	9.3	2.20
Small	>10	42	0.054	4.7	1.54
Cobble					
Large	>5	42	0.054	2.3	1.08
Small	>2.5	41	0.052	1.1	0.75
Gravel					
Very coarse	>1.3	40	0.050	0.54	0.52
Coarse	>0.6	38	0.047	0.25	0.36
Medium	>0.3	36	0.044	0.12	0.24
Fine	>0.16	35	0.042	0.06	0.17
Very fine	>0.08	33	0.039	0.03	0.12
Sands					
Very coarse	>0.04	32	0.029	0.01	0.070
Coarse	>0.02	31	0.033	0.006	0.055
Medium	>0.01	30	0.048	0.004	0.045
Fine	>0.005	30	0.072	0.003	0.040
Very fine	>0.003	30	0.109	0.002	0.035
Silts					
Coarse	>0.002	30	0.165	0.001	0.030
Medium	>0.001	30	0.25	0.001	0.025

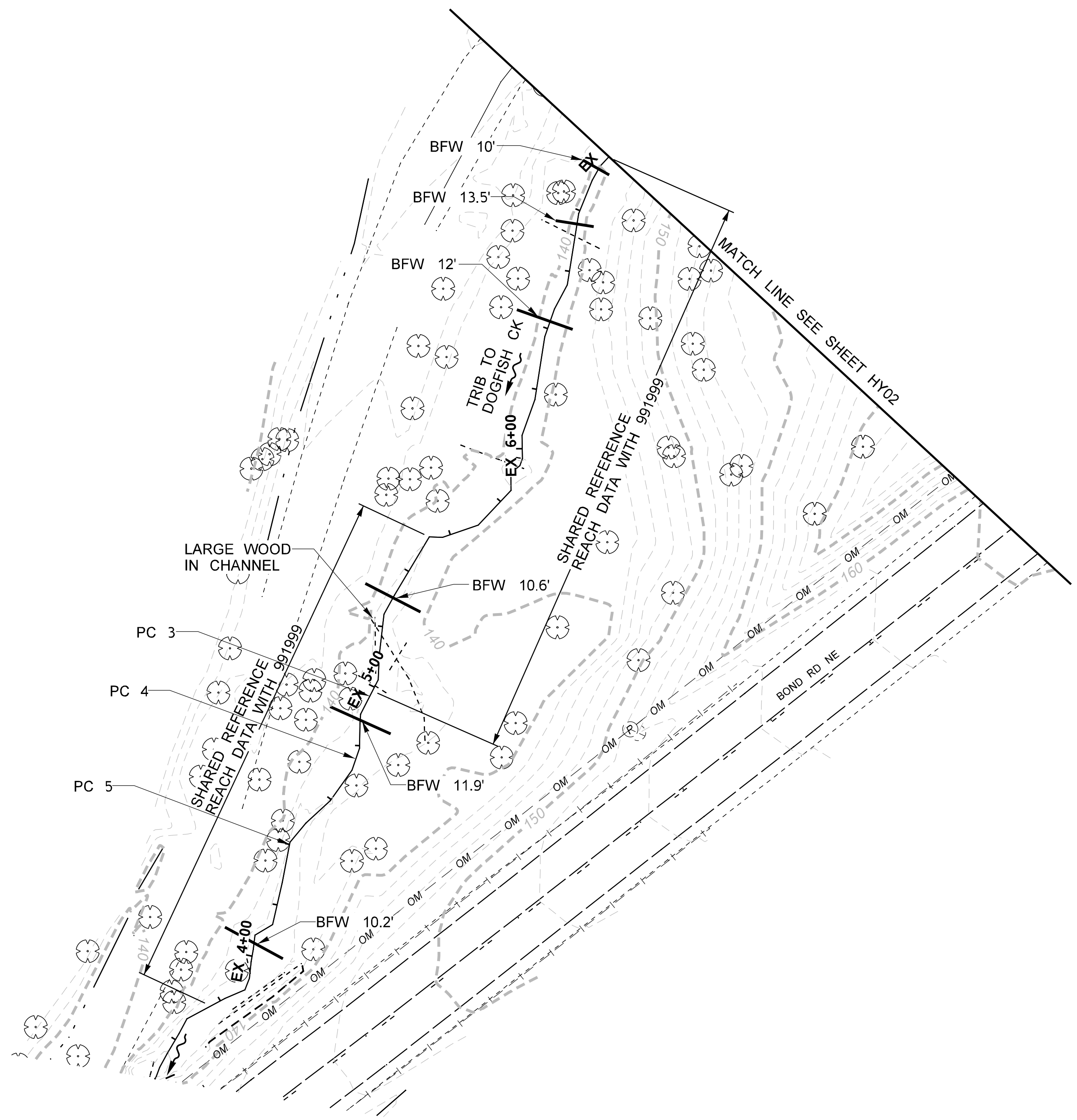
https://dec.vermont.gov/sites/dec/files/wsm/rivers/docs/assessment-protocol-appendices/O-Appendix-O-04-Shear-Stress.pdf
https://www.engr.colostate.edu/~pierre/ce_old/classes/ce717/Manuals/Fischenich/Fischenich%202001.pdf

Appendix C: Streambed Material Sizing Calculations

LEGEND

- MAJOR CONTOUR (350)
- MINOR CONTOUR
- EDGE OF PAVEMENT
- THALWEG
- RD MRK EDGE WHITE LINE
- GUARD RAIL
- LINE NO PASS
- OVERHEAD UTILITIES MULTIPLE (OM)
- LOG
- DITCH BOTTOM
- TREE
- SIGN
- POWER POLE
- GUY ANCHOR
- EX 4+00 EXISTING ALIGNMENT
- PR 1+00 PROPOSED ALIGNMENT

NOTE:
(1) SITE SURVEY, STREAM GRADING AND EXISTING CONTOURS BASED ON SURVEY COMPLETED BY WSDOT FEBRUARY 2022 (NAVD 88) .



N

DATUM (NAVD) 88

0 25 50
SCALE IN FEET

PRELIMINARY

NOT FOR CONSTRUCTION

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DESIGNED BY				
ENTERED BY				
CHECKED BY				
PROJ. ENGR.				
REGIONAL ADM.		REVISION	DATE	BY

REGION NO. 10 | STATE WASH | FED.AID PROJ.NO.
JOB NUMBER
CONTRACT NO. | LOCATION NO.

P.E. STAMP BOX DATE
P.E. STAMP BOX DATE

Washington State Department of Transportation

NE DOGFISH CREEK
SR 307 MP 1.45
991572 IMPROVEMENT PROJECT
PRELIMINARY DESIGN

EXISTING STREAM PLAN

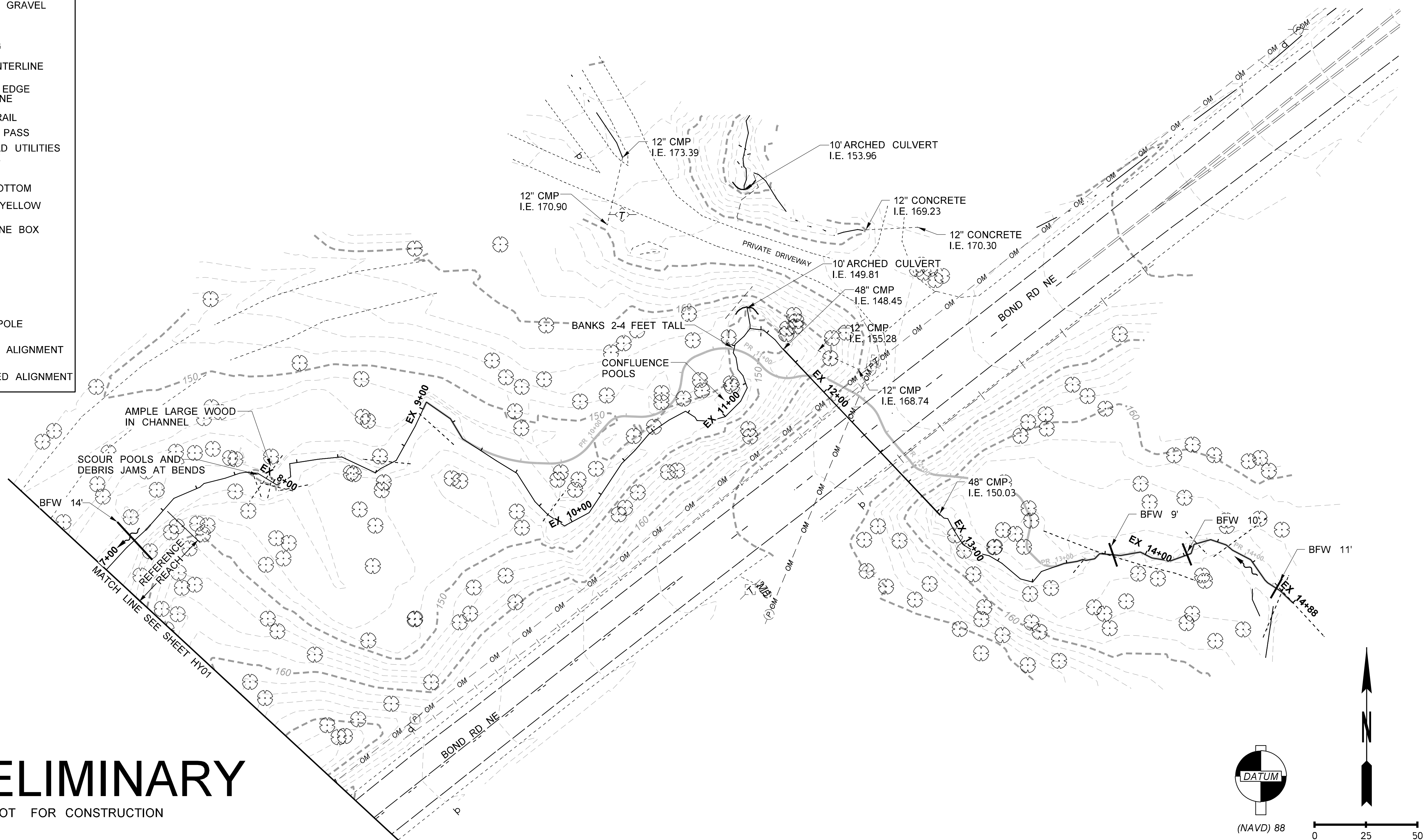
PLAN REF NO HY01

SHEET 1 OF 5 SHEETS

LEGEND

- MAJOR CONTOUR
- MINOR CONTOUR
- EDGE OF PAVEMENT
- EDGE OF GRAVEL ROAD
- CULVERT
- THALWEG
- SKIP CENTERLINE
- RD MRK EDGE WHITE LINE
- GUARD RAIL
- LINE NO PASS
- OVERHEAD UTILITIES MULTIPLE
- LOG
- DITCH BOTTOM
- DOUBLE YELLOW LINE
- TELEPHONE BOX
- MAILBOX
- TREE
- SIGN
- POWER POLE
- EX 4+00 EXISTING ALIGNMENT
- PR 1+00 PROPOSED ALIGNMENT

NOTE:
(1) SITE SURVEY, STREAM GRADING AND EXISTING CONTOURS BASED ON SURVEY COMPLETED BY WSDOT FEBRUARY 2022 (NAVD 88).



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DESIGNED BY			
ENTERED BY			
CHECKED BY			
PROJ. ENGR.			
REGIONAL ADM.	REVISION	DATE	BY

REGION NO. 10 | STATE WASH | FED.AID PROJ.NO.

JOB NUMBER

CONTRACT NO. | LOCATION NO.

P.E. STAMP BOX DATE

P.E. STAMP BOX DATE

Washington State Department of Transportation

NE DOGFISH CREEK
SR 307 MP 1.45
991572 IMPROVEMENT PROJECT
PRELIMINARY DESIGN

EXISTING STREAM PLAN

PLAN REF NO HY02

SHEET 2 OF 5 SHEETS

LEGEND

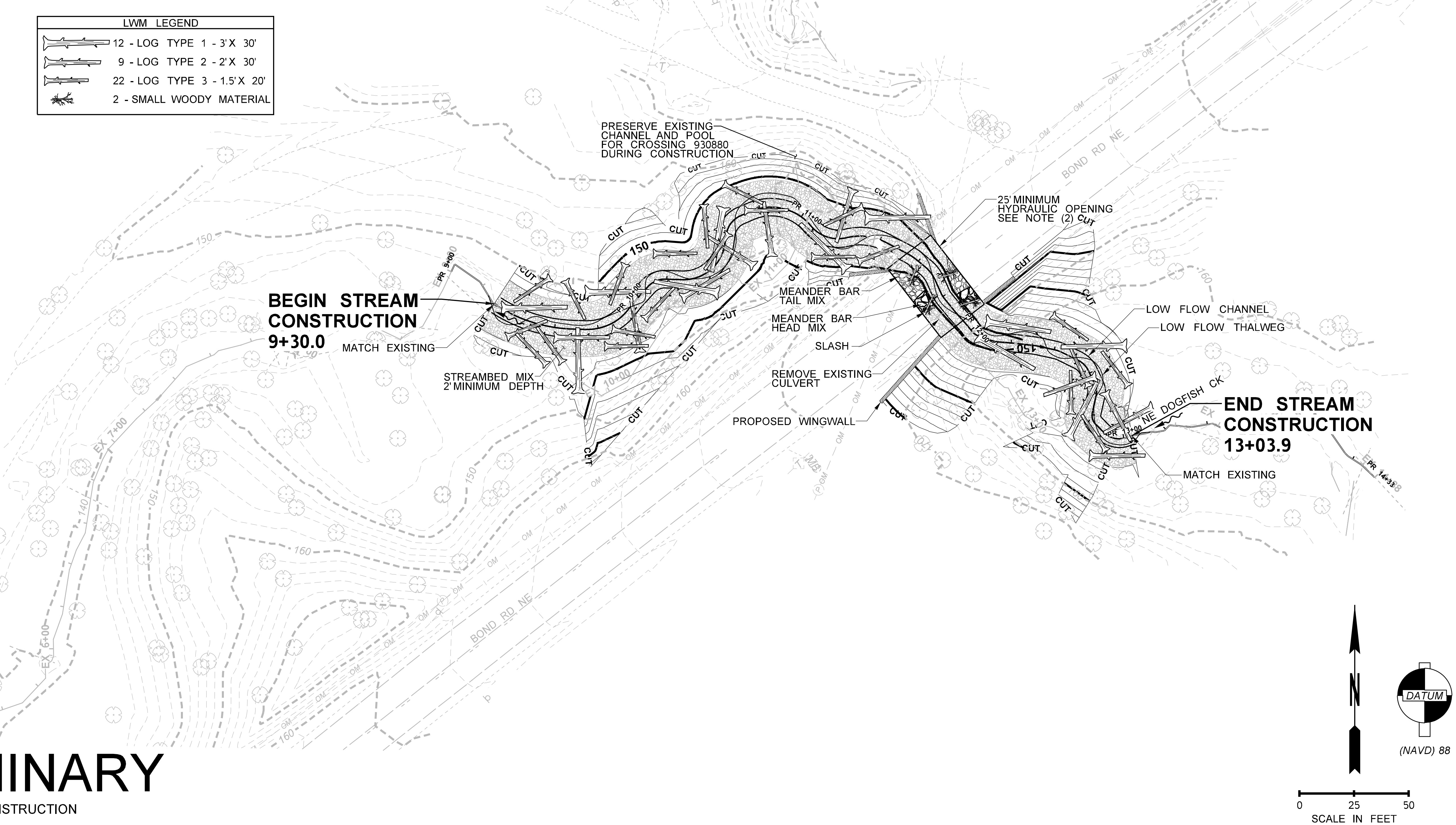
350	PROPOSED MAJOR CONTOUR
	PROPOSED MINOR CONTOUR
350	MAJOR CONTOUR
	MINOR CONTOUR
	EDGE OF PAVEMENT
	EDGE OF GRAVEL ROAD
	EXISTING STRUCTURE
	RD MRK EDGE WHITE LINE
	GUARD RAIL
	LINE NO PASS
OM — OM	OVERHEAD UTILITIES MULTIPLE
	LOG
	DITCH BOTTOM
	DOUBLE YELLOW LINE
T	TELEPHONE BOX
MB	MAILBOX
	TREE
	SIGN
	POWER POLE
	GUY ANCHOR
EX 4+00	EXISTING ALIGNMENT
PR 1+00	PROPOSED ALIGNMENT
CUT	NEW CUT LINE
	WINGWALL

NOTES:

(1) EXISTING GROUND PROFILE SAMPLED DIGITAL TERRAIN MODEL DEVELOPED FROM WSDOT FIELD SURVEY FEBRUARY 2022 (NAVD 88).

(2) EXACT STRUCTURE TYPE, SIZE, LOCATION, AND WALLS TO BE DETERMINED.

(3) GRADING LIMITS SHOWN ARE FOR ILLUSTRATION PURPOSES ONLY. FINAL LIMITS TO BE DETERMINED BASED ON FINAL STRUCTURE, TYPE, SIZE, AND LOCATION.



PRELIMINARY

NOT FOR CONSTRUCTION

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TIME	10:09:16 AM			
DATE	2/17/2023			
PLOTTED BY	MikeL			
DESIGNED BY				
ENTERED BY				
CHECKED BY				
PROJ. ENGR.				
REGIONAL ADM.		REVISION	DATE	BY

REGION NO.	STATE	FED.AID PROJ.NO.
10	WASH	
JOB NUMBER		
CONTRACT NO.		LOCATION NO.

P.E. STAMP BOX DATE

P.E. STAMP BOX DATE

Washington State Department of Transportation

NE DOGFISH CREEK
SR 307 MP 1.45
991572 IMPROVEMENT PROJECT
PRELIMINARY DESIGN

PROPOSED CONDITIONS PLAN

PLAN REF NO HY03

SHEET 3 OF 5 SHEETS

NOTES:

(1) EXISTING GROUND PROFILE SAMPLED DIGITAL TERRAIN MODEL DEVELOPED FROM WSDOT FIELD SURVEY FEBRUARY 2022 (NAVD 88).

(2) MATERIAL DEPTH IS APPROXIMATE. FINAL DEPTH OF STREAMBED DEPTH AND MINIMUM FOUNDATION DEPTH TO BE DETERMINED FOLLOWING SCOUR ANALYSIS.

(3) SLOPES SHOWN OUTSIDE OF THE MINIMUM CHANNEL SECTION ARE FOR ILLUSTRATIVE PURPOSES ONLY TO DEPICT ESTIMATED AREA OF POTENTIAL IMPACT. FINAL AREAS OF IMPACT TO BE DETERMINED PENDING GEOTECHNICAL AND STRUCTURAL INVESTIGATION, STRUCTURE TYPE, AND STRUCTURE LOCATION.

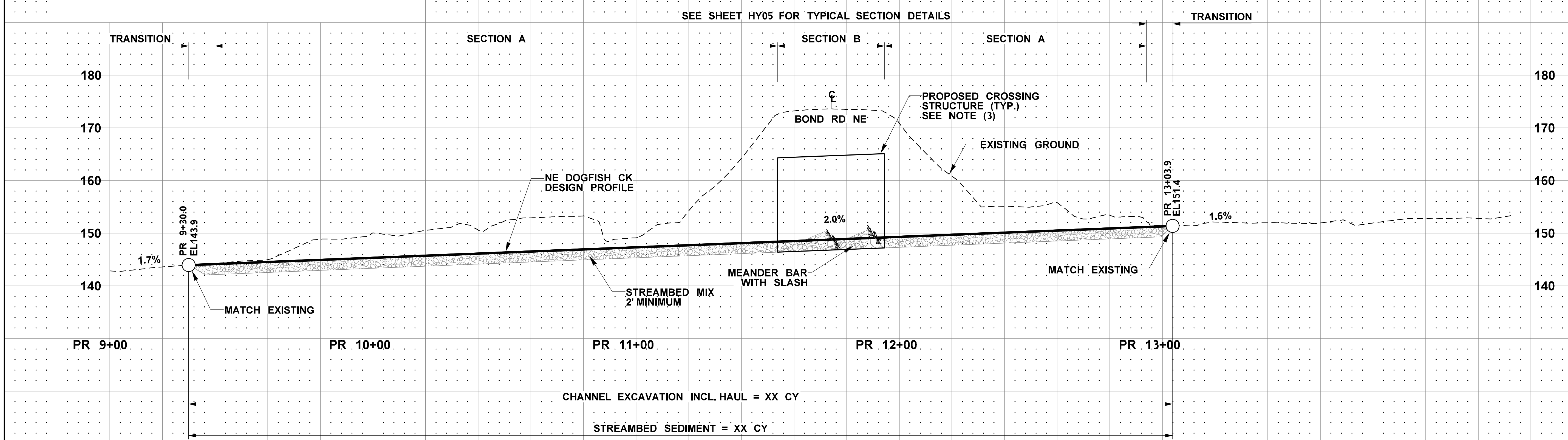
(4) 25 FEET IS BANK TO BANK LIMIT, MIN. HYDRAULIC OPENING WITHIN CROSSING IS 25 FEET.

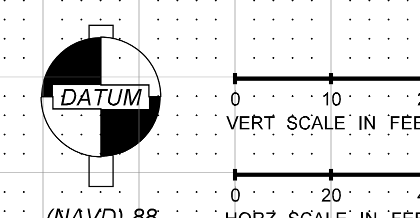
(5) EXACT STRUCTURE TYPE, SIZE, LOCATION, AND WALLS TO BE DETERMINED. ALL STRUCTURAL ELEMENTS INCLUDING ANY SOIL RELIED UPON FOR LATERAL FOUNDATION SUPPORT, MUST BE OUTSIDE OF STRUCTURE FREE ZONE.

(6) SEE SPECIAL PROVISION "AGGREGATES FOR STREAMS, RIVERS, AND WATERBODIES" FOR STREAMBED MATERIAL AND MATERIAL LIFTS.

(7) FROM PR 9+30 TO PR 9+40, EVENLY TAPER SECTION A TO MATCH EXISTING CHANNEL.

(8) FROM PR 12+94 TO PR 13+04, EVENLY TAPER SECTION A TO MATCH EXISTING CHANNEL.





PRELIMINARY

NOT FOR CONSTRUCTION

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PLOTTED BY	MikeL			
DESIGNED BY				
ENTERED BY				
CHECKED BY				
PROJ. ENGR.	-			
REGIONAL ADM.	-	REVISION	DATE	BY

REGION NO. 10 | STATE WASH | FED.AID PROJ.NO.

JOB NUMBER

CONTRACT NO. 14C510 | LOCATION NO.

P.E. STAMP BOX DATE

P.E. STAMP BOX DATE

Washington State Department of Transportation

NE DOGFISH CREEK

SR 307 MP 1.45

991572 IMPROVEMENT PROJECT

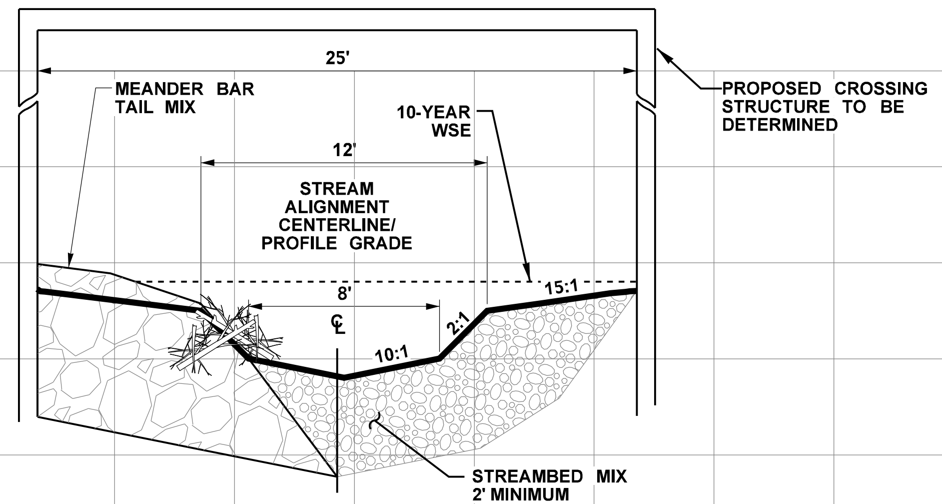
PRELIMINARY DESIGN

STREAM PROFILE

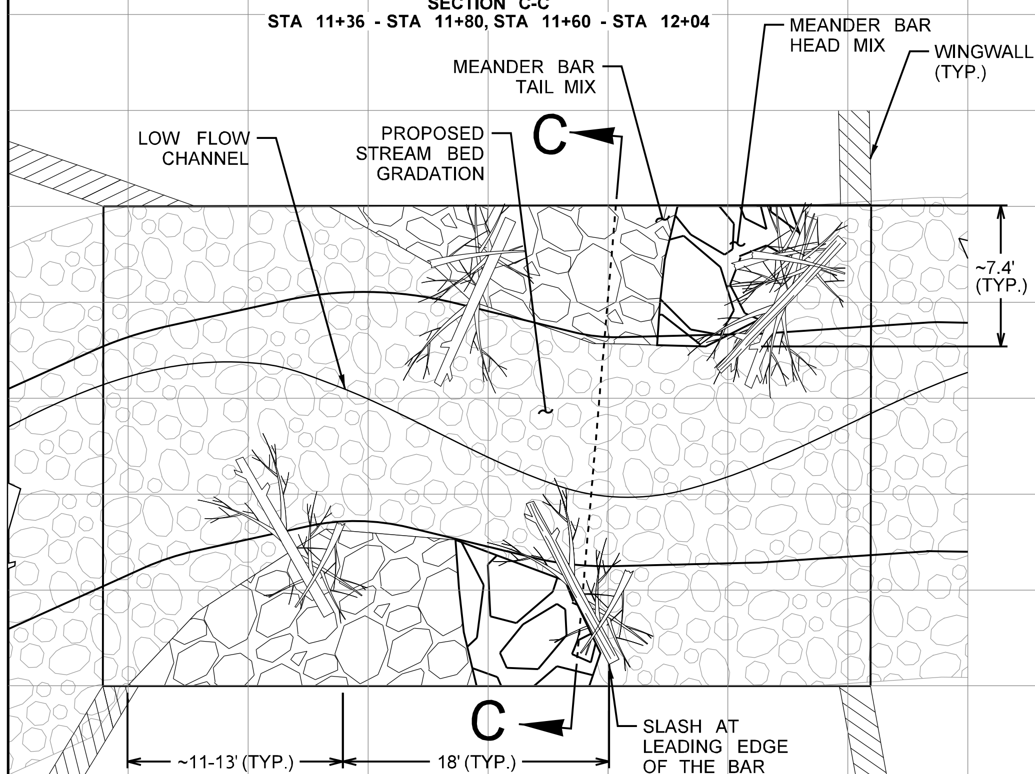
PLAN REF. NO. HY04

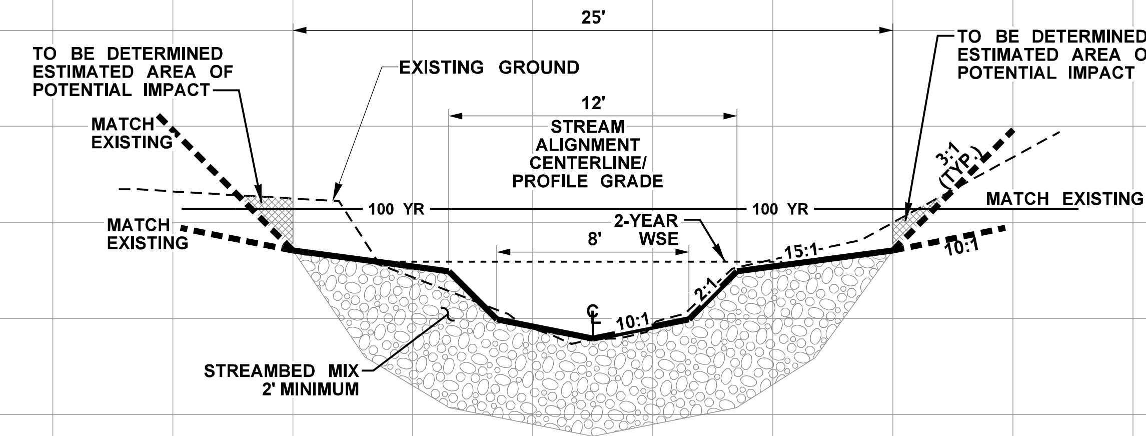
SHEET 4 OF 5 SHEETS

NOTES:
(1) CHANNEL BED DEPTH IS PRELIMINARY AND IS SHOWN FOR ILLUSTRATIVE PURPOSES ONLY. MATERIAL QUANTITY AND DEPTH TO BE DETERMINED FOLLOWING A SCOUR ANALYSIS CONDUCTED AT FHD DESIGN PHASE.
(2) SLOPES SHOWN OUTSIDE OF THE MINIMUM CHANNEL SECTION ARE FOR ILLUSTRATIVE PURPOSES ONLY TO DEPICT ESTIMATED AREA OF POTENTIAL IMPACT. FINAL AREAS OF IMPACT TO BE DETERMINED PENDING GEOTECHNICAL AND STRUCTURAL INVESTIGATION, STRUCTURE TYPE, AND STRUCTURE LOCATION.
(3) EXACT STRUCTURE TYPE, SIZE, LOCATION, AND WALLS TO BE DETERMINED. ALL STRUCTURAL ELEMENTS INCLUDING ANY SOIL RELIED UPON FOR LATERAL FOUNDATION SUPPORT, MUST BE OUTSIDE OF THE STRUCTURE FREE ZONE.
(4) SEE SPECIAL PROVISION "AGGREGATES FOR STREAMS, RIVERS, AND WATERBODIES" FOR STREAMBED MATERIAL AND MATERIAL LIFTS.
(5) ACCOMMODATING STRUCTURE FREE ZONE REQUIRES CONFIRMATION OF STRUCTURE, PAVEMENT AND COVER THICKNESS.
(6) A MAINTENANCE CLEARANCE OF 10 FEET TO ALLOW FOR MACHINERY TO ACCESS AND OPERATE UNDER THE STRUCTURE IS REQUIRED.
(7) FROM PR 9+30 TO PR 9+40, EVENLY TAPER SECTION A TO MATCH EXISTING CHANNEL.
(8) FROM PR 12+94 TO PR 13+04, EVENLY TAPER SECTION A TO MATCH EXISTING CHANNEL.



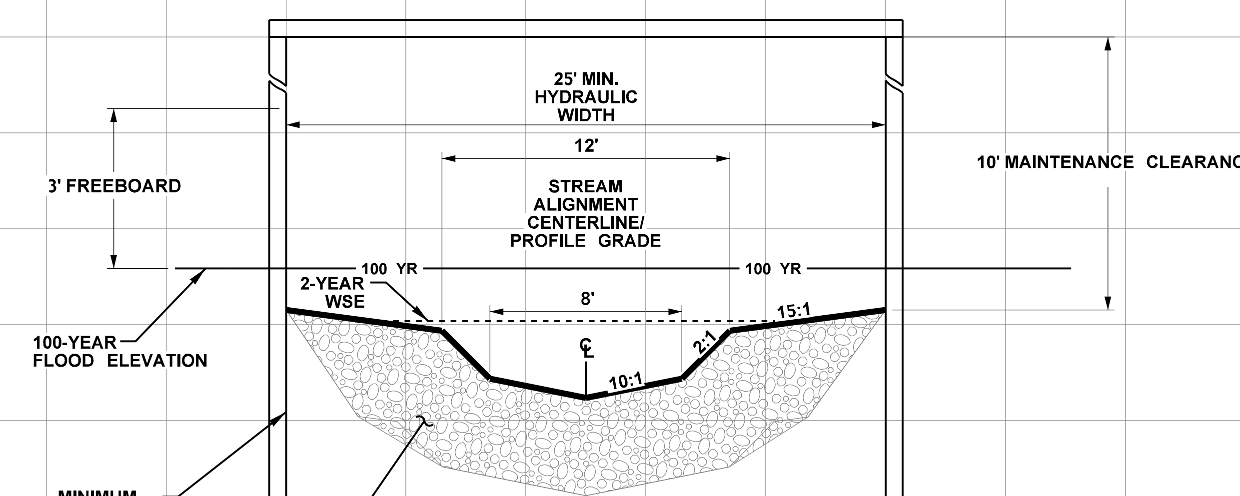
TYPICAL MEANDER BAR SECTIONS
SECTION C-C
STA 11+36 - STA 11+80, STA 11+60 - STA 12+04





NE DOGFISH CREEK
TYPICAL SECTION
LOOKING DOWNSTREAM
SECTION A
STA 9+30 - STA 11+36
STA 12+04 - 13+03.86

PROPOSED CROSSING STRUCTURE TO BE DETERMINED



NE DOGFISH CREEK
TYPICAL STRUCTURE SECTION
LOOKING DOWNSTREAM
SECTION B
STA 11+36 - STA 12+04

DATUM
(NAVD) 88

0 2 4
VERT SCALE IN FEET

0 5 10
HORZ SCALE IN FEET

PRELIMINARY
NOT FOR CONSTRUCTION

0 5 10
SCALE IN FEET

FILE NAME	C:\Fish Passaages\PHD 991572 UNT TO DOGFISH CREEK\CADD\ENGINEERING\Sheets\PHD 991572 UNT TO DOGFISH CREEK_PR_HY_05.dgn			
TIME	10:53:09 AM			
DATE	2/17/2023			
PLOTTED BY	MikeL			
DESIGNED BY				
ENTERED BY				
CHECKED BY				
PROJ. ENGR.	-			
REGIONAL ADM.	-	REVISION	DATE	BY

REGION NO.	STATE	FED.AID PROJ.NO.
10	WASH	
JOB NUMBER		
CONTRACT NO.	LOCATION NO.	
14C510		

DATE
P.E. STAMP BOX

DATE
P.E. STAMP BOX

Washington State
Department of Transportation

NE DOGFISH CREEK
SR 307 MP 1.45
991572 IMPROVEMENT PROJECT
PRELIMINARY DESIGN
STREAM TYPICAL SECTIONS

PLAN REF. NO.
HY05

SHEET
5
OF
5
SHEETS

Appendix D: Stream Plan Sheets, Profile, Details

Stream Name: 991572 Tributary to Dogfish Creek **Reach:** Existing Channel
Stream Slope, S (ft/ft): 0.01750 **Date:** 4/16/2022
Practitioner: TVM

Reach D_{50}, D_{84} (mm): **Step D_{84} (mm)[(a)]:**
Hydraulic Radius, R (ft):
Mean Flow Depth, d (ft)[(b)]:
Bedform Variation, σ_z (ft)[(c)]:
Median Thalweg Depth, h_m (ft)[(c)]:
Large Wood in Steps? (y/n)[(c)]:

Notes:
(a) Required for Lee and Ferguson (2002) method, for step-pool streams (S>0.027)
(b) Mean flow depth = hydraulic depth; Required for Bathurst (1985), Rickenmann and Recking (2011), and Aberle and Smart (2003) methods
(c) Longitudinally; Provide for S>~0.03 ft/ft (see sheet "S>0.03, Sigma z")

1 **Consult Tabular Guidance** → 2 **Consult Photographic Guidance** → 3 **Apply a Quantitative Prediction Method**

Flow resistance in stream channels is due to roughness induced by bed and bank grain material, bedforms (such as dunes and step pools), planform, vegetation, large instream wood, and other obstructions. Flow resistance coefficient estimation (Manning's n, Darcy-Weisbach f) is approximate, requiring redundancy (steps 1 through 3) for confidence in the implimented values. Dependence on quantitative methods alone is not recommended since utilized reaches in the derivisions were intentionally selected to have little influence from sinuosity, instream large wood, streambank vegetation, bank irregularities, obstructions, etc.; these types of flow resistance are not lumped into the quantitative estimates. Also, flow resistance coefficients should be computed at the flow magnitude of interest for the objectives of the analysis, specifically at high, bankfull, or low flow.

1 Tabular Guidance

Sources: Brunner (2016): pp 3-14
Arcement and Schneider (1989): p 4
Aldridge and Garrett (1973): p 24

Note: Key references are provided in the spreadsheet package zip file or are available for download through the links provided in the references of the supporting technical summary report (TS-103).

2 Photographic Guidance

Sources: USGS (online photo guidance)
Yochum et al. (2014): high gradient
Hicks and Mason (1991)
Aldridge and Garrett (1973)
Barnes (1967)

	n	f	Use in Average? Enter "y"
Tabular Estimate:	0.050	----	y
Estimate from Photographic Guidance:	0.057	----	y

Instructions: **(See technical summary report, TS-103, for more detailed instructions and references.)**

(1) Grey cells indicate fields that should be populated. Results are provided in the salmon colored cells.
(2) Enter background information (cells D4, D5, I4 to I6), sediment size data (cells D8, E8, H8), and hydraulic information (cells D9 to D13). R is often approximated as the average depth for steams with a width/depth ratio > ~20.
(3) Consult tabular guidance and enter the best estimate in the grey box (cell I43; do not use in average if not confident of estimate). Tabular values are typically substantially underestimated for channels > ~3% slope.
(4) Consult photographic guidance and enter an estimate in the grey box (cell I44).
(5) Applicable quantitative procedures will be automatically compute (per provided Applicable Range).
(6) Implement Arcement and Schneider (1989) procedure, if desired (cells T20 to Y20).

U.S. Forest Service
National Stream and Aquatic Ecology Center
Tool developed by: Steven E. Yochum, PhD, PE, Hydrologist
Tool reviewed by: Julian A. Scott, Hydrolgist

Stream Name: 991572 Tributary to Dogfish Creek **Reach:** Existing Channel
Slope, S (ft/ft): 0.01750 **Date:** 4/16/2022
Practitioner: TVM

D_{50}, D_{84}, $D_{84,\ step}$ (m):	----	----	----
R (ft, m):	----	----	
d (ft^2, m^2):	----	----	
σ_z (ft, m):	----	----	
h_m (ft, m):	----	----	

Overall Average n:	0.054
f:	----
Quantitative Average n (1):	----
f (1):	----
Arcement and Schneider (1989) n:	0.054
f:	----

3 Quantitative Prediction

Quasi-Quantitative:

	n_b (4)	n_1	n_2	n_3	n_4	m	Estimate	Use in Average? Enter "y"
Arcement and Schneider (1989) $n = (n_b + n_1 + n_2 + n_3 + n_4)m$	0.03	0.005	0.007	0.01	0.002	1	0.054	y
	Base	Degree of Irrigularity	Variation in X-S	Effect of Obstruction	Amount of Vegetation	Degree of Meandering		

Fully Quantitative:

Method [Fit]	Relative Submergence	Estimate n	Estimate f	# Data Points	Applicable Range Slope (ft/ft)	Applicable Range Relative Sub.(3)	Use in Average? Enter "y"
Yochum et al. (2012) [R^2 = 0.78; f: R^2 = 0.82]	----	----	----	78	0.02 to 0.20	h_m/σ_z = 0.25 to 12	
Rickenmann and Recking (2011)	----	----	----	2890	0.00004 to 0.03	d/D_{84} = 0.18 to ~100	
Aberle and Smart (2003); in flume	----	----	----	94	0.02 to 0.10	d/σ_z = 1.2 to 12	
Lee and Ferguson (2002)(4) [RMS error = 19%]	----	----	----	81	0.027 to 0.184	R/D_{84}(step) = 0.1 to 1.4	
Bathurst (1985) [RMS error = ~34%]	----	----	----	44	0.00429 to 0.0373	d/D_{84} = 0.71 to 11.4	
Jarrett (1984) [ave. std. error = 28%]	n/a	----	----	75	0.002 to 0.039	n/a	
Griffiths (1981); rigid bed [R^2=0.59]	----	----	----	84	0.000085 to 0.011	R/D_{50} = 1.8 to 181	
Hey (1979); a = 12.72	----	----	----	30	0.00049 to ~0.01	R/D_{84} = 0.8 to 25	
Limerinos (1970) [R^2=0.77]	----	----	----	50	0.00038 to 0.039	R/D_{84} = 1.1 to 69	

Notes:
(1) Quantitative average excludes the Arcement and Schneider (1989) method.
(2) In some situations it can be appropraite to assume that the quantitative average n is n_b., though this may result in overestimated flow resistance.
(3) Relative submergence is computed using either R (hydraulic radius) or d (mean depth) and the D_{50} (median bed material size) or D_{84} (84% of bed material smaller); or computed using either h_m (median thalweg depth) or d and σ_z (standard deviation of residuals of a thalweg longitudinal profile regression). For σ_z computation, see "S>0.03, Sigma z" tab of this spreadsheet.
(4) This method can substantially underestimate flow resistance in steeper streams (slope>0.03) where large wood is present and incorporated into the steps, enhancing step heights.

This spreadsheet has been reviewed for accuracy. However, the ultimate responsibility for flow resistance estimates remains with the user.
U.S. Forest Service
National Stream and Aquatic Ecology Center

Figure 4A-2 Manning's Roughness Coefficients for Stream Channels

Stream Channels	Manning's n
Minor streams (surface width at flood stage less than 100 feet):	
1. Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030-0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035-0.05
c. Some weeds, light brush on banks	0.035-0.05
d. Some weeds, heavy brush on banks	0.05-0.07
e. Some weeds, dense willows on banks	0.06-0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by 0.01-0.02	
2. Irregular sections, with pools, slight channel meander; increase values given in 1a-e above 0.01-0.02	
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04-0.05
b. Bottom of cobbles, with large boulders	0.05-0.07
Floodplains (adjacent to natural streams):	
1. Pasture, no brush:	
a. Short grass	0.030-0.035
b. High grass	0.035-0.05
2. Cultivated areas:	
a. No crop	0.03-0.04
b. Mature row crops	0.035-0.045
c. Mature field crops	0.04-0.05
3. Heavy weeds, scattered brush	0.05-0.07
4. Light brush and trees:	
a. Winter	0.05-0.06
b. Summer	0.06-0.08
5. Medium to dense brush:	
a. Winter	0.07-0.11
b. Summer	0.10-0.16
6. Dense willows, summer, not bent over by current	0.15-0.20
7. Cleared land with tree stumps, 100 to 150 per acre:	
a. No sprouts	0.04-0.05
b. With heavy growth of sprouts	0.06-0.08
8. Heavy stand of timber, a few down trees, little under-growth:	
a. Flood depth below branches	0.10-0.12
b. Flood depth reaches branches	0.12-0.16

MAIN CHANNEL TABULAR ESTIMATE USED IN SPREADSHEET

OVERBANK AREA

Major streams (surface width at flood stage more than 100 feet): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of n may be somewhat reduced. Follow recommendation in publication cited if possible. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of 0.028-0.033.

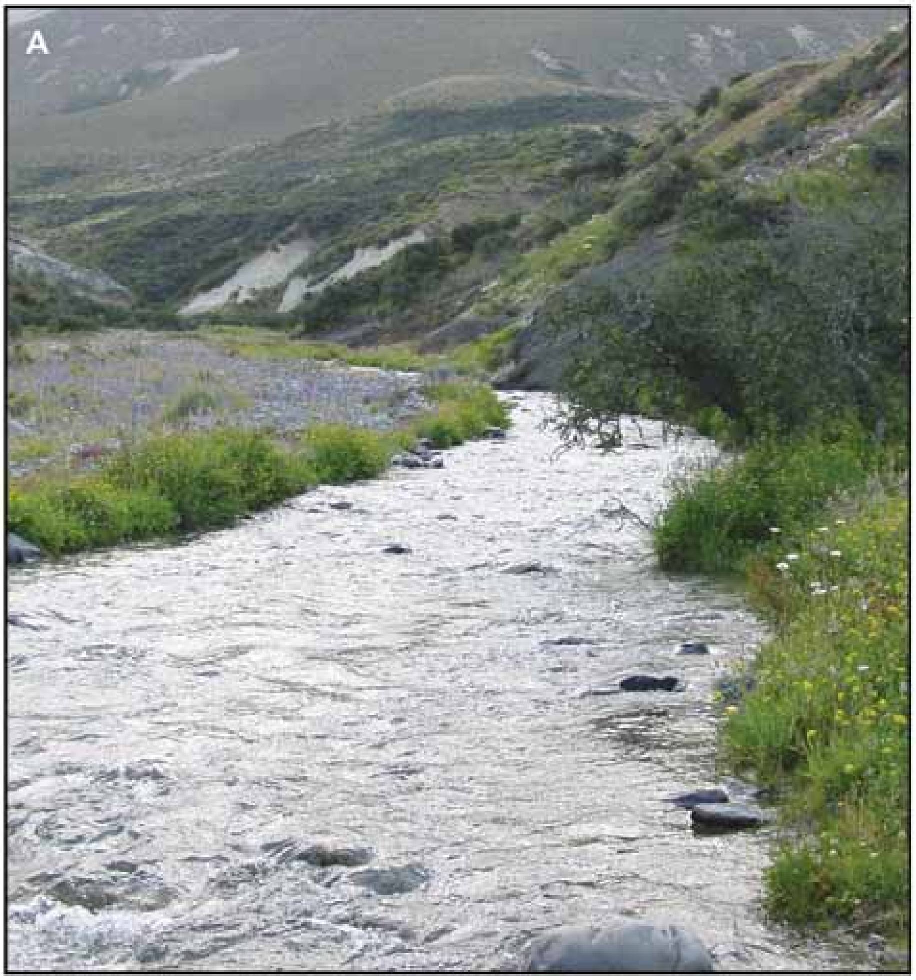
Porter-11 (plane-bed)

Porter River, South Island, New Zealand

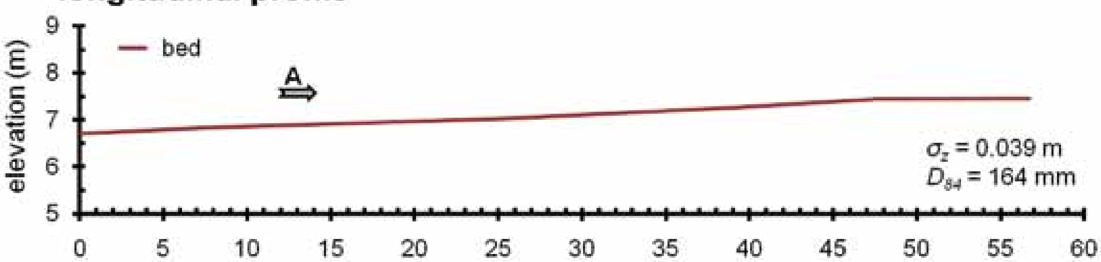
S = 0.013 m/m; W = 5.1 m (17 ft); L = 57 m (186 ft), stream classification (Rosgen): C3

n = 0.057	V = 0.68 m/s (2.2 ft/s)
f = 0.44	R = 0.2 m (0.66 ft)

1/16/2003



longitudinal profile



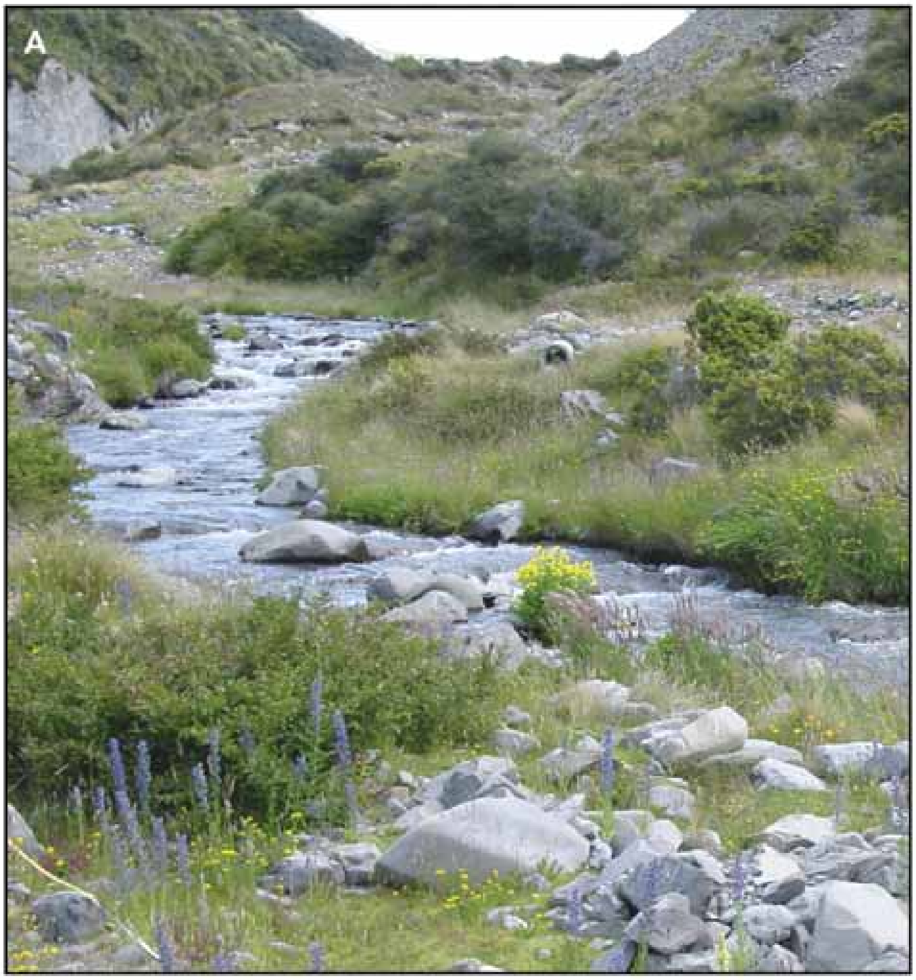
Meander bar overbank flow

Porter-6 (transitional: plane-bed / step-pool)

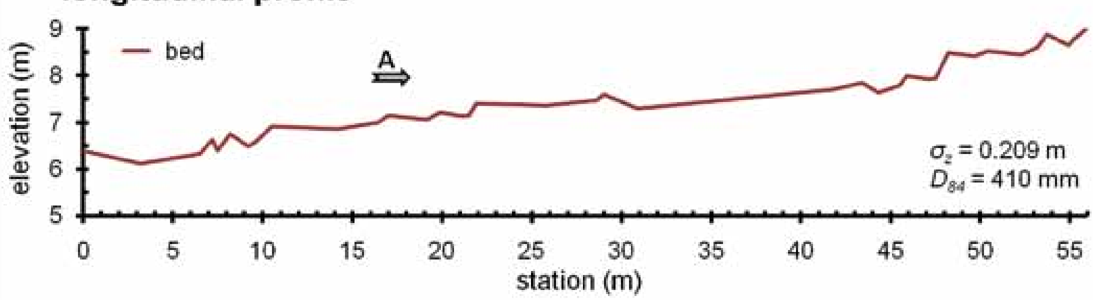
Porter River, South Island, New Zealand
S = 0.047 m/m; W = 3.3 m (11 ft); L = 56 m (183 ft), stream classification (Rosgen): B3a

n = 0.091	V = 0.81 m/s (2.7 ft/s)
f = 1.1	R = 0.2 m (0.66 ft)

1/16/2003



longitudinal profile



Appendix E: Manning's Calculations

WSDOT Large Woody Material for stream restoration metrics calculator

State Route# & MP	***SR 307 MP 1.45***		**Key piece volume**	***1.310***	*yd3*
Stream name	*NE Dogfish Creek*		**Key piece/ft**	***0.0335***	*per ft stream*
length of regrade[a]	*374*	*ft*	**Total wood vol./ft**	***0.3948***	*yd3/ft stream*
Bankfull width	*12*	*ft*	**Total LWM**[c] **pieces/ft stream**	***0.1159***	*per ft stream*
Habitat zone[b]	*Western WA*				

Log type	***Diam at midpoint ****	***Length*** [d]	***Volume/log*** [d]	***Rootwad?***	***Qualifies as key piece?***	***No. LWM pieces***	***Total wood volume***
	ft	**ft**	**yd3**				**yd3**
1	3.00	*30*	7.85	**yes**	yes	11	86.39
2	2.00	*30*	3.49	**yes**	yes	10	34.91
3	1.50	*20*	1.31	**yes**	no	22	28.80
4			0.00				0.00
5			0.00				0.00
6			0.00				0.00
7			0.00				0.00
8			0.00				0.00
9			0.00				0.00
10			0.00				0.00

	No. of key pieces	**Total No. of LWM pieces**	**Total LWM volume (yd3)**
Design	21	43	150.1
75% Targets	**13**	**43**	**147.7**
50% Targets	**7**	**33**	**76.1**
	surplus	on target	surplus

Appendix F: Large Woody Material Calculations

Future Projections for Climate-Adapted Culvert Design

Project Name:	991572
Stream Name:	Unnamed Trib to Dogfish Creek
Drainage Area:	645 ac

Projected mean percent change in bankfull flow:

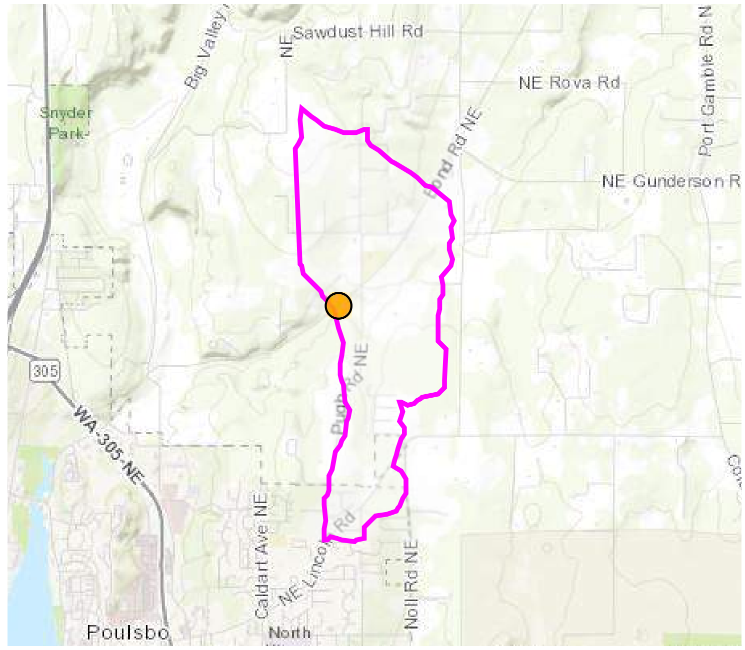
2040s:	13.8%
2080s:	16.9%

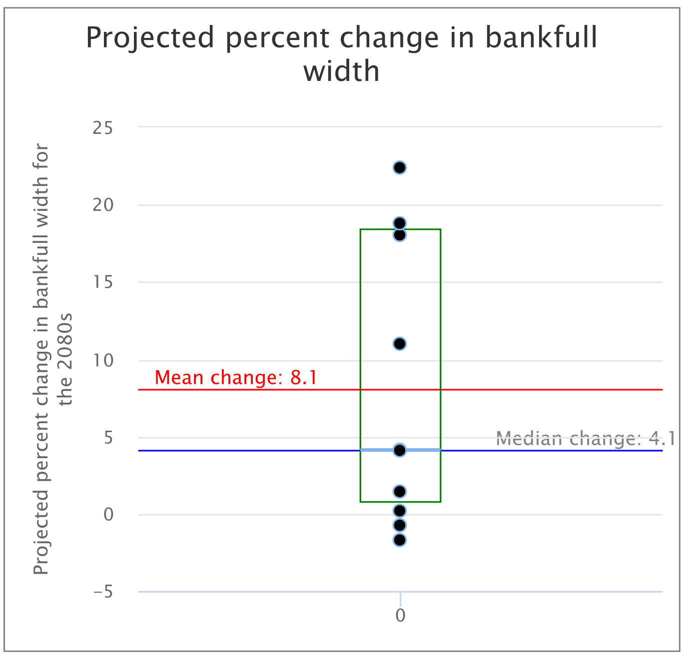
Projected mean percent change in bankfull width:

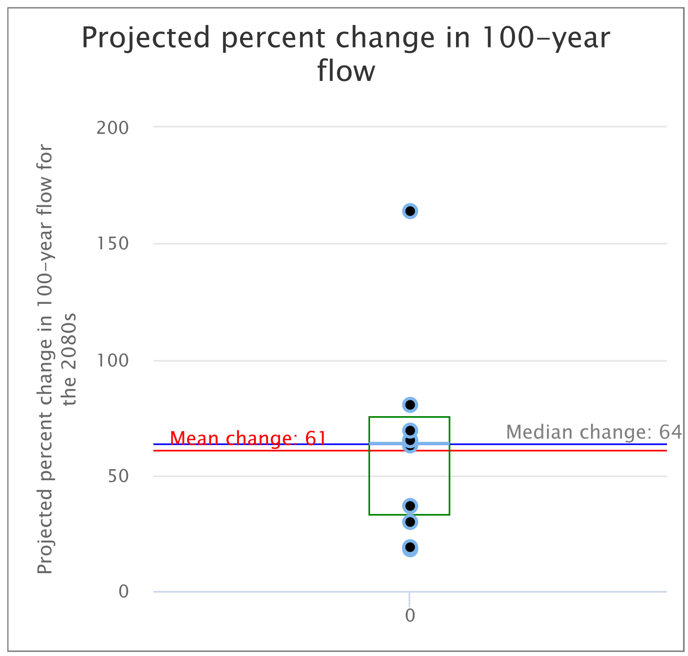
2040s:	6.7%
2080s:	8.1%

Projected mean percent change in 100-year flood:

2040s:	42.4%
2080s:	61%







Black dots are projections from 10 separate models

The Washington Department of Fish and Wildlife makes no guarantee concerning the data's content, accuracy, precision, or completeness. WDFW makes no warranty of fitness for a particular purpose and assumes no liability for the data represented here.

Appendix G: Future Projections for Climate-Adapted Culvert Design

Appendix H: SRH-2D Model Results

Fig 1. Existing Channel Profile

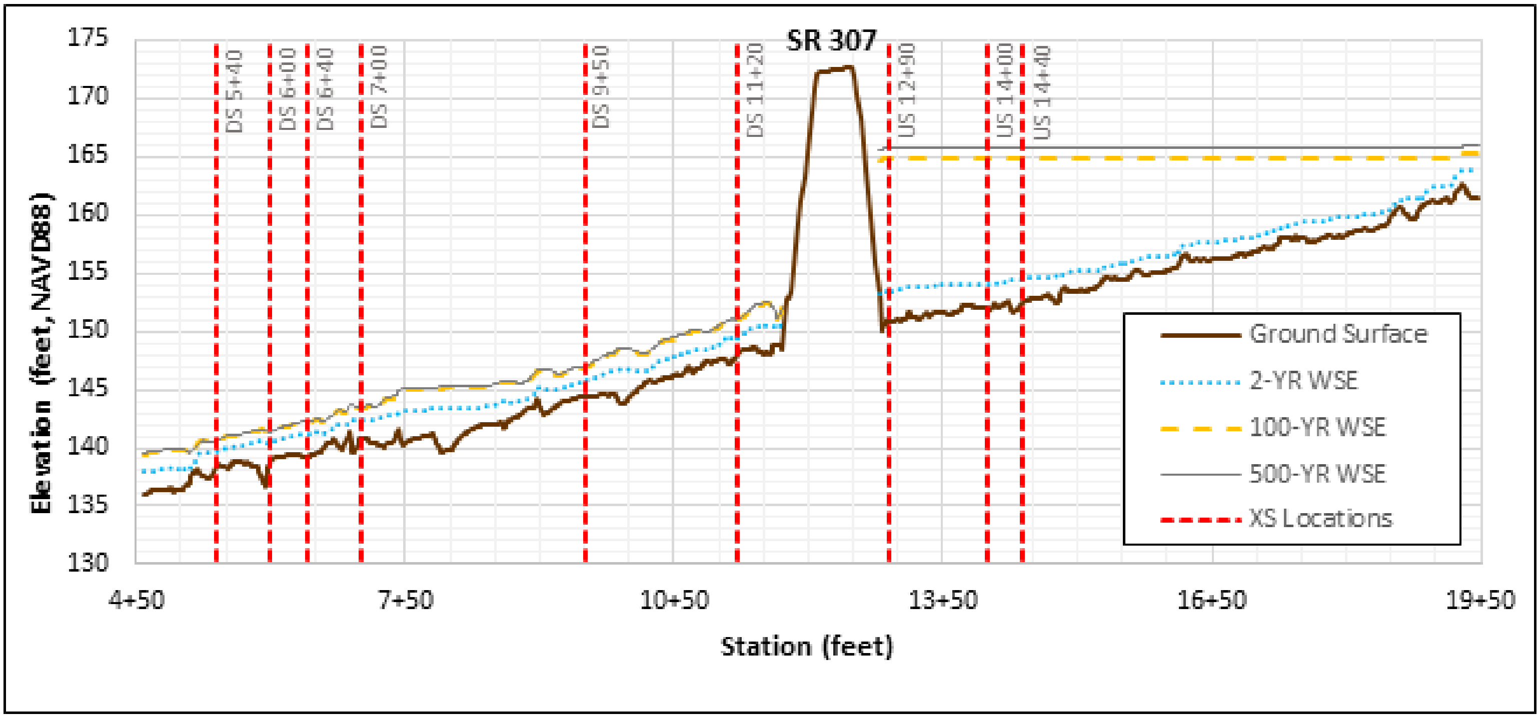


Fig 2. Natural Channel Profile

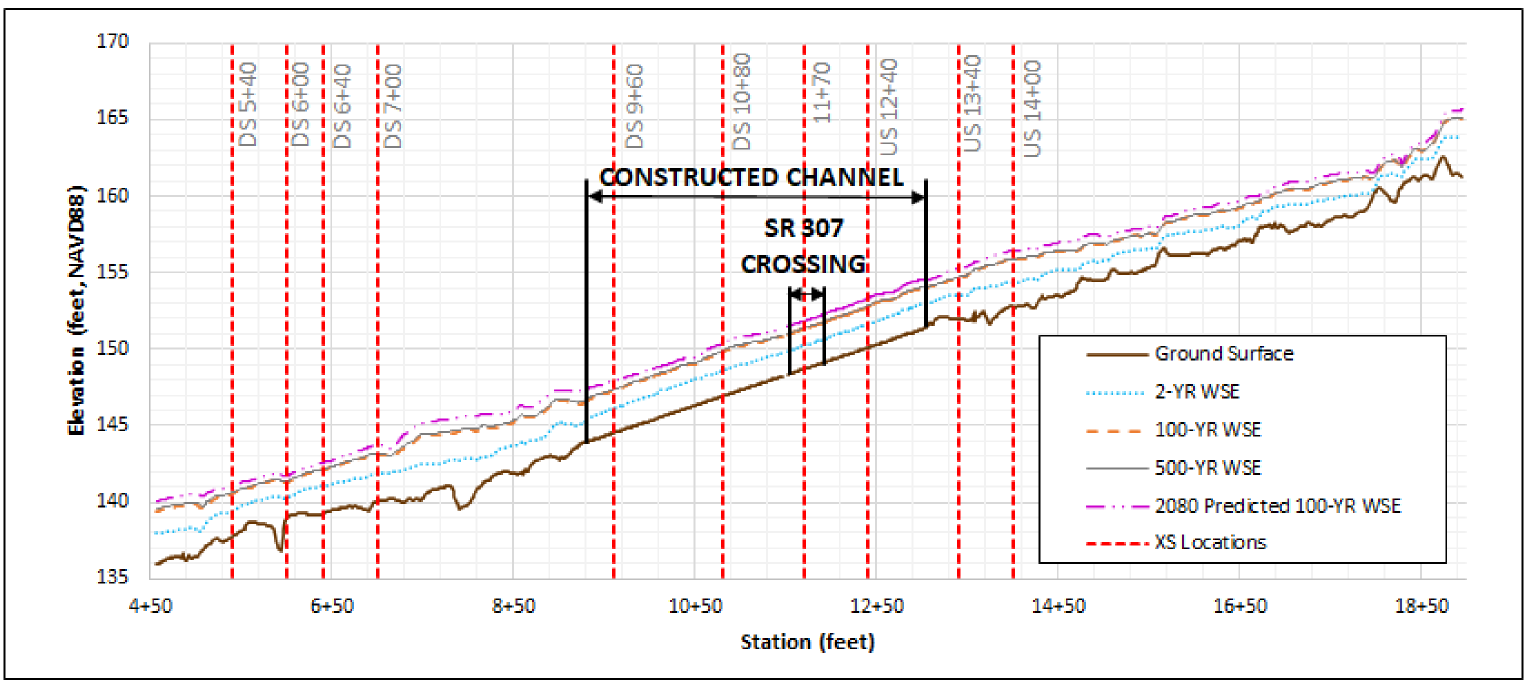


Fig 3. Proposed Channel Profile

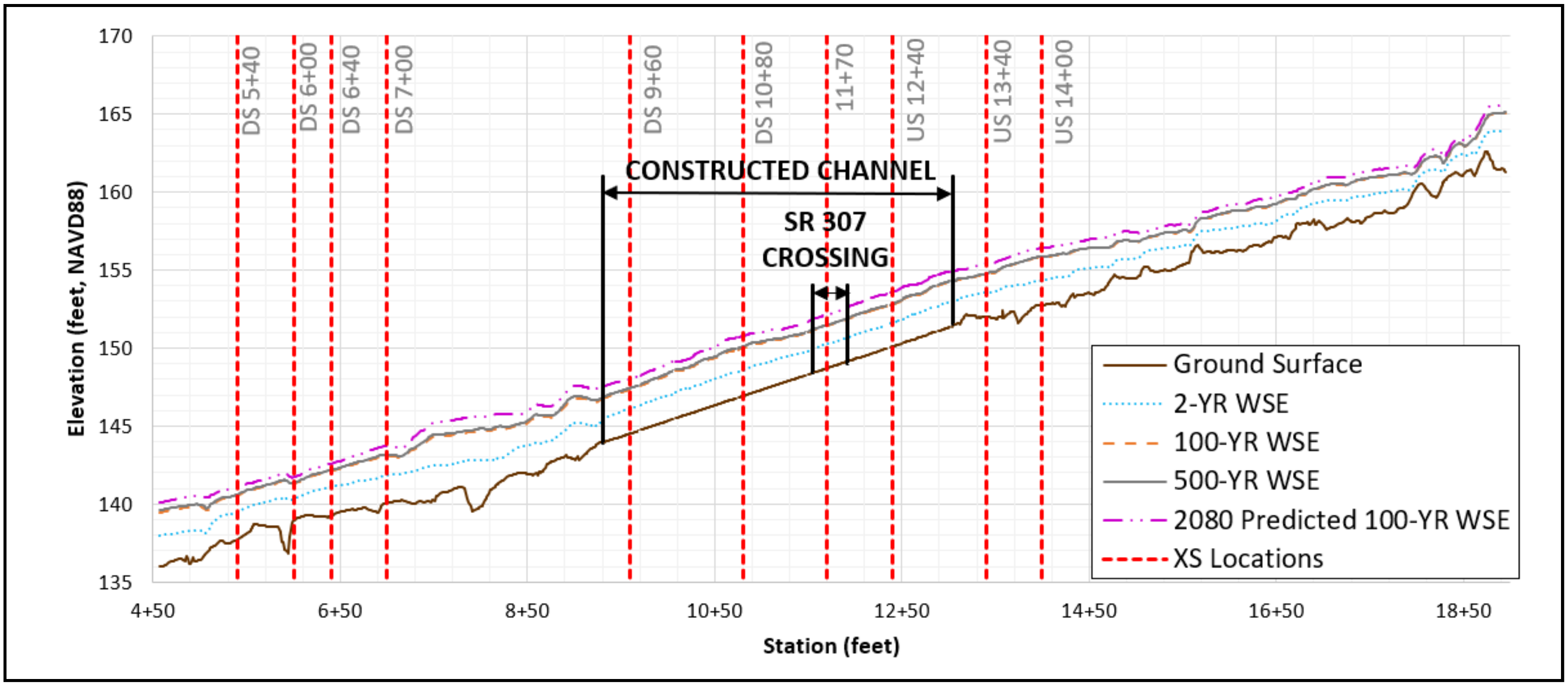


Fig 4. Existing Downstream Cross-Section A EX 5+40

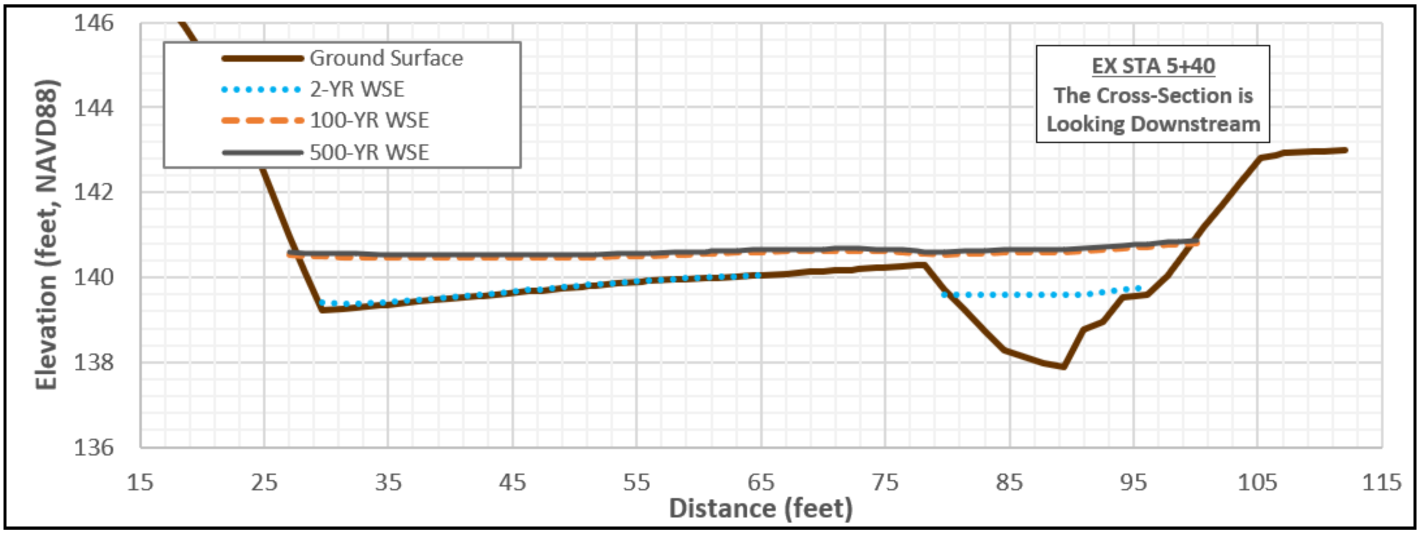


Fig 5. Natural Downstream Cross-Section A PR 5+40

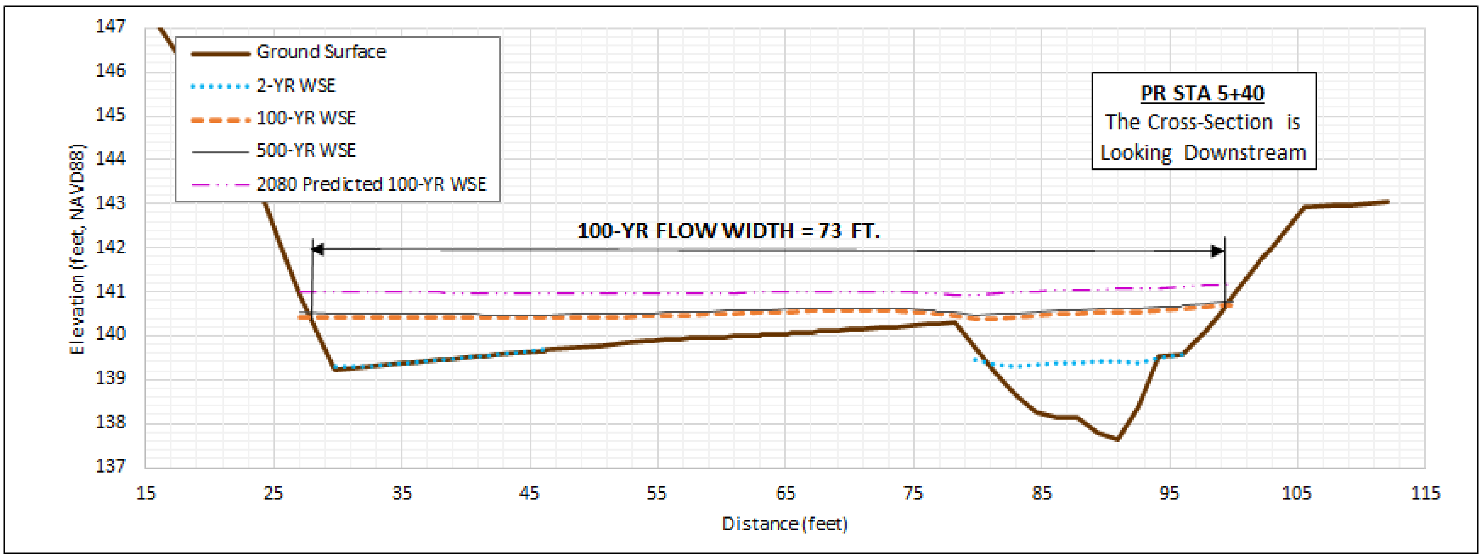


Fig 6. Proposed Downstream Cross-Section A PR 5+40

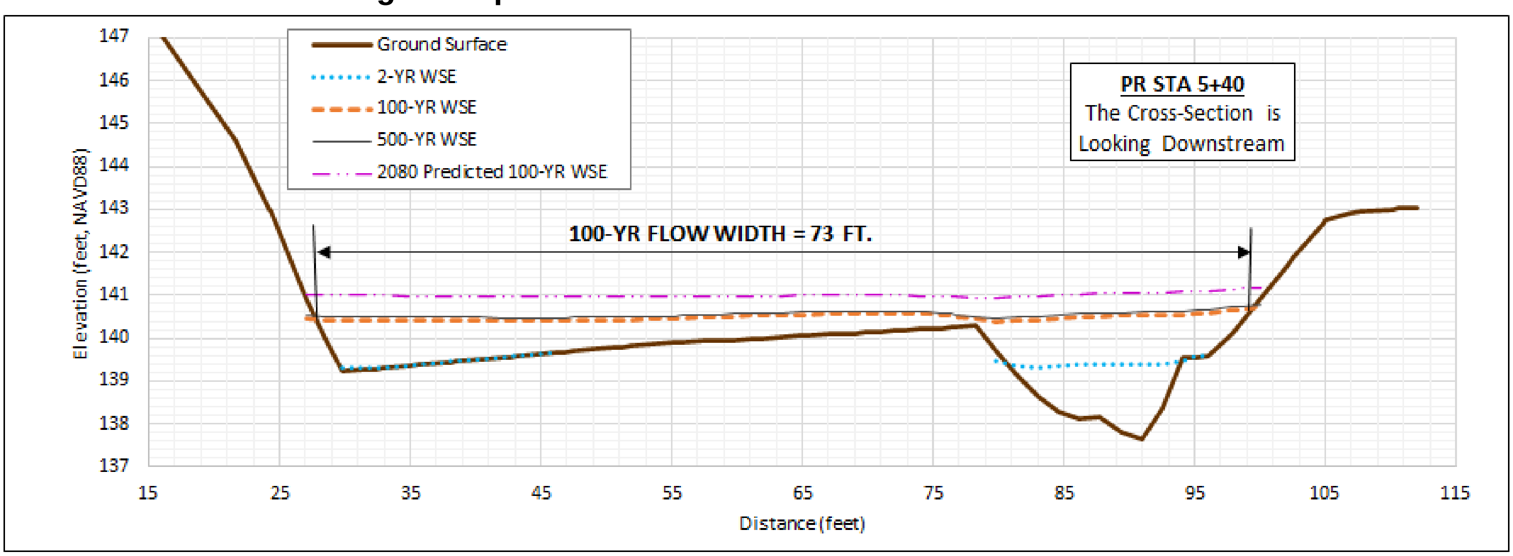


Fig 7. Existing Downstream Cross-Section B EX 6+00

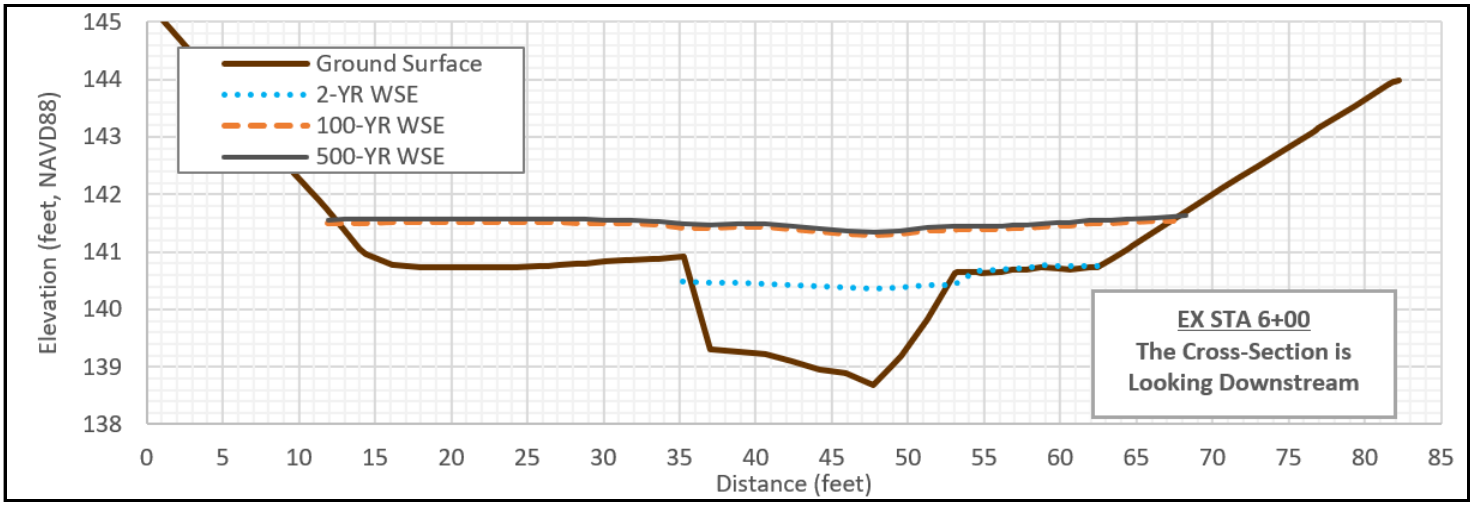


Fig 8. Natural Downstream Cross-Section B PR 6+00

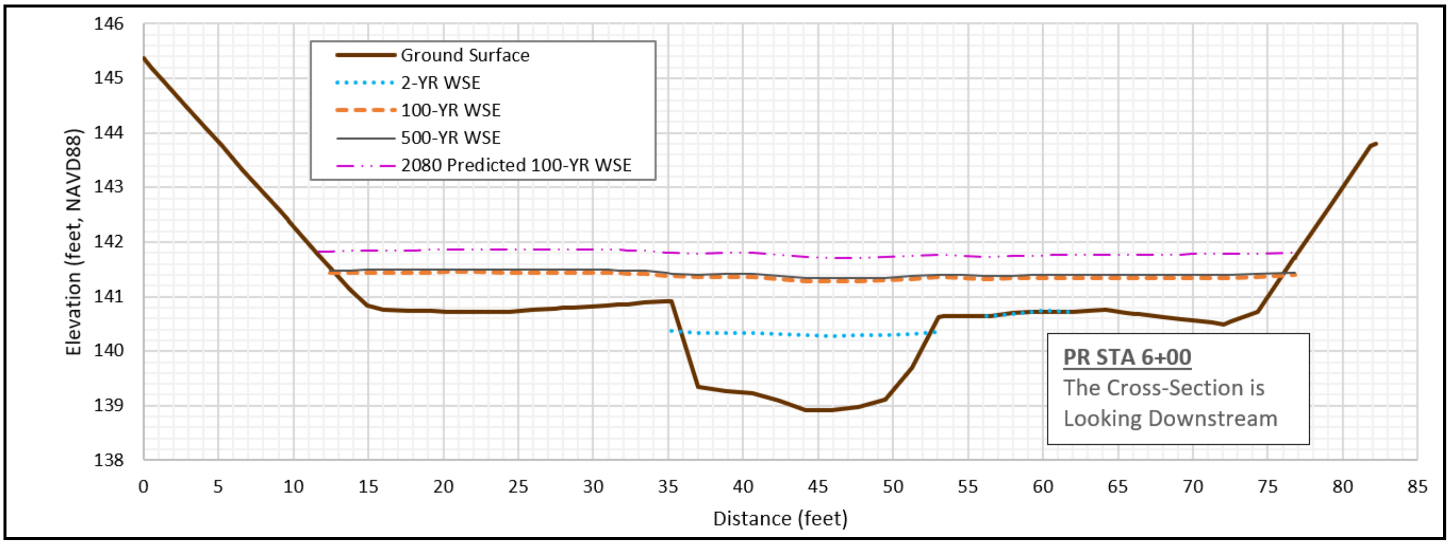


Fig 9. Proposed Downstream Cross-Section B PR 6+00

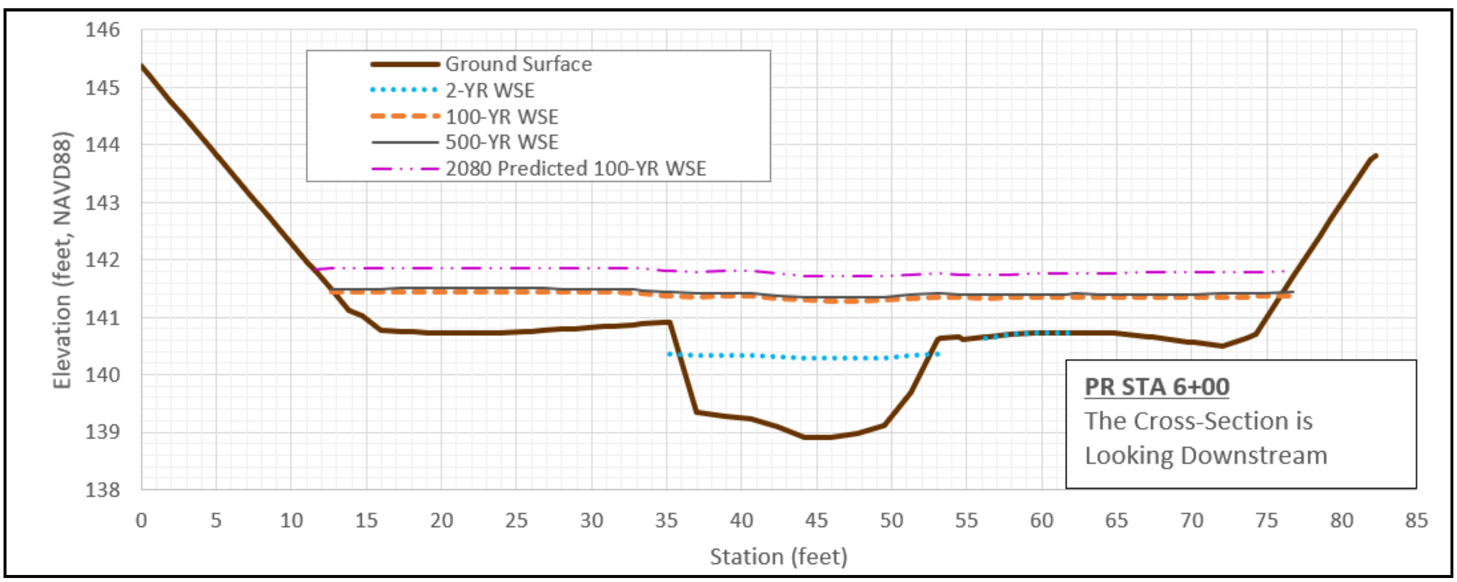


Fig 10. Existing Downstream Cross-Section C EX 6+40

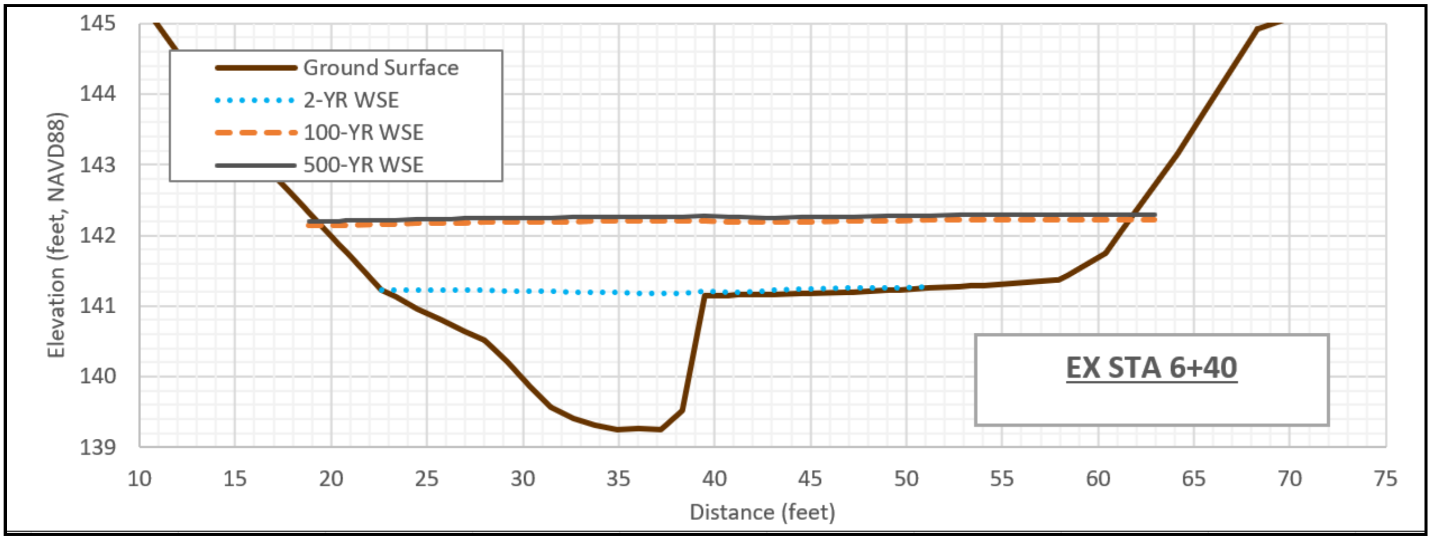


Fig 11. Natural Downstream Cross-Section C PR 6+40

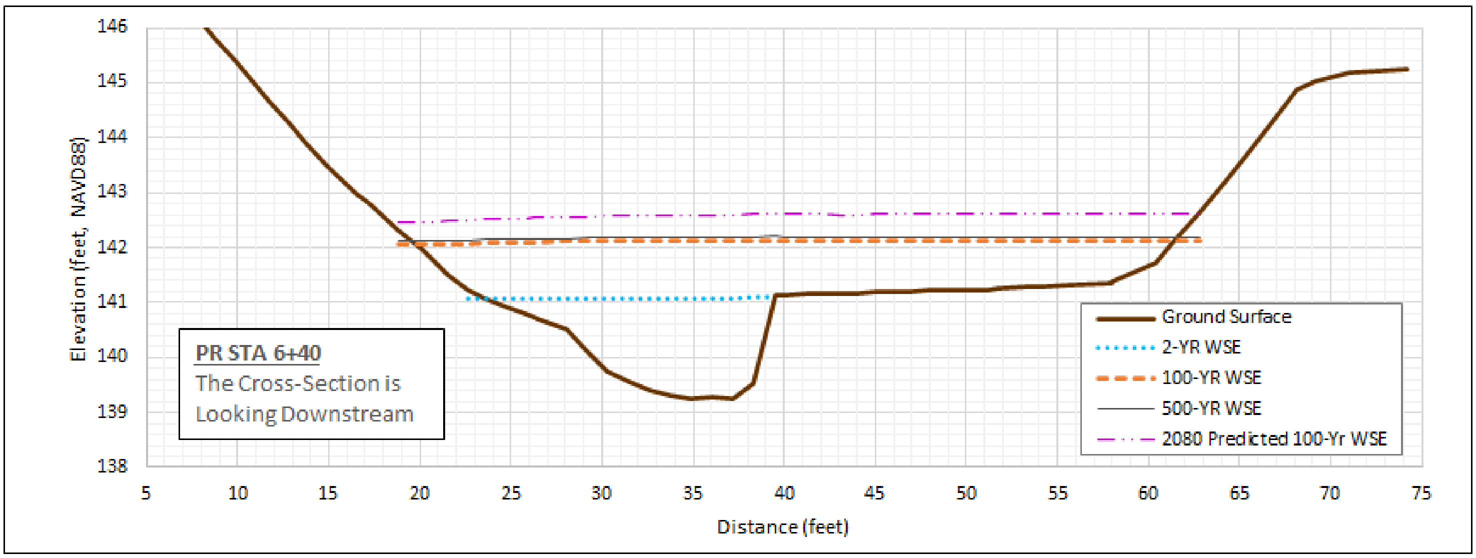


Fig 12. Proposed Downstream Cross-Section C PR 6+40

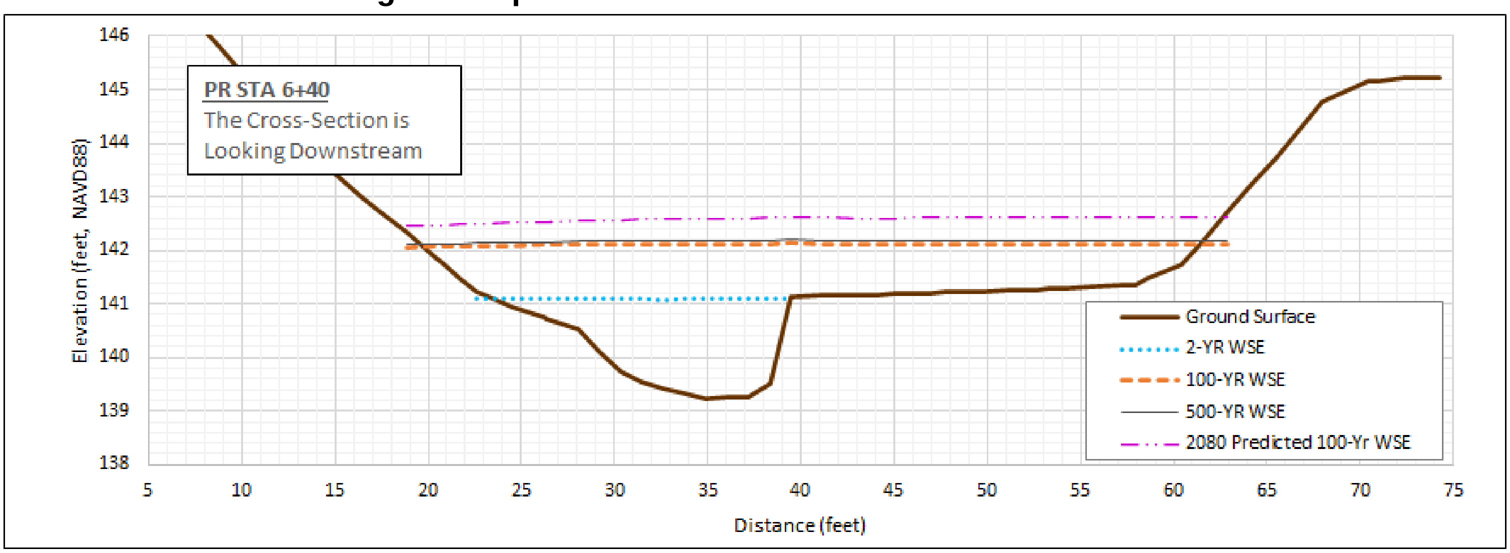


Fig 13. Existing Downstream Cross-Section D EX 7+00

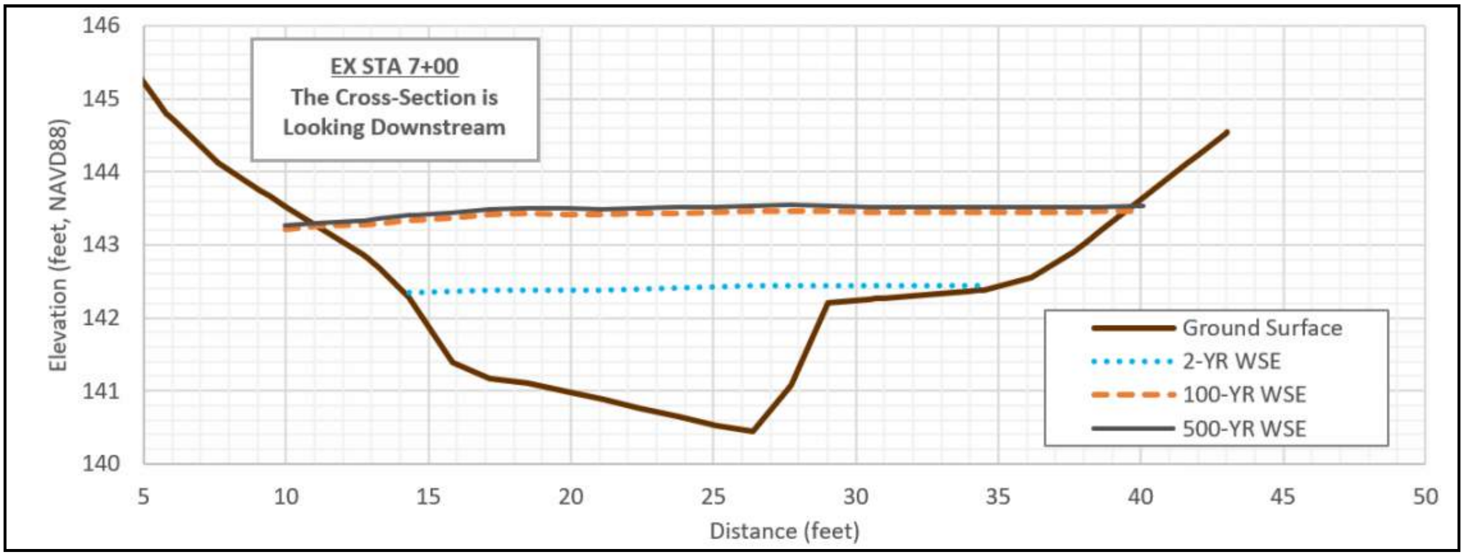


Fig 14. Natural Downstream Cross-Section D PR 7+00

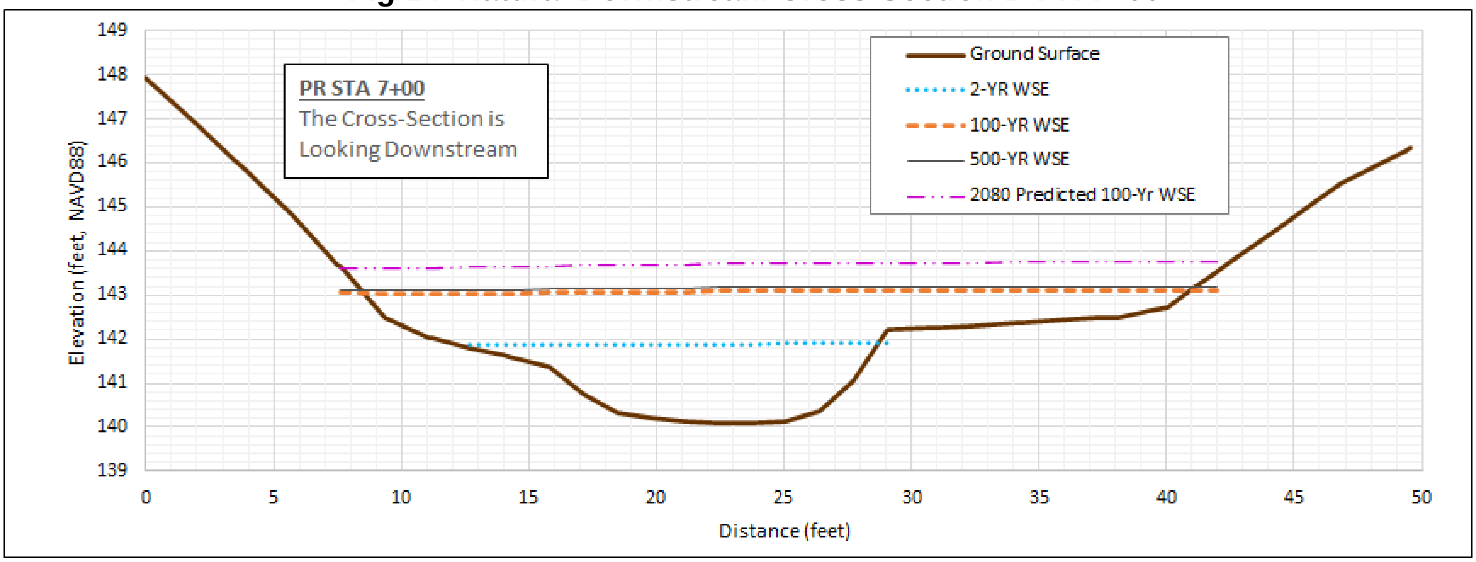


Fig 15. Proposed Downstream Cross-Section D PR 7+00

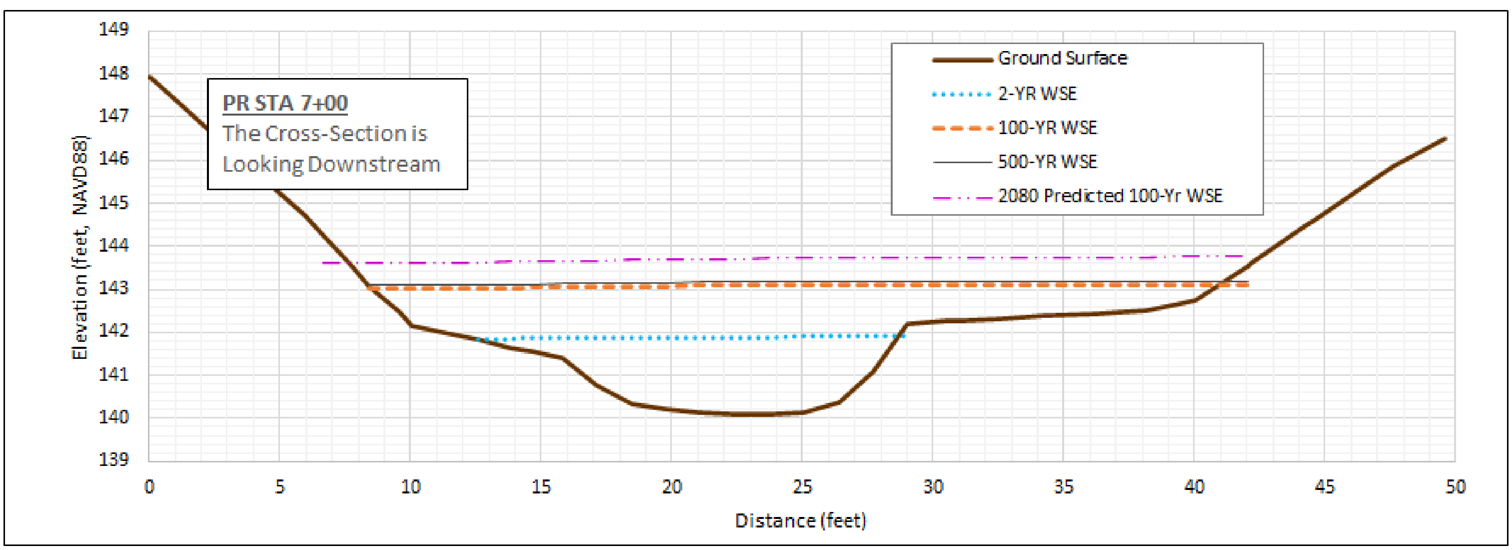


Fig 16. Existing Downstream Cross-Section E EX 9+50

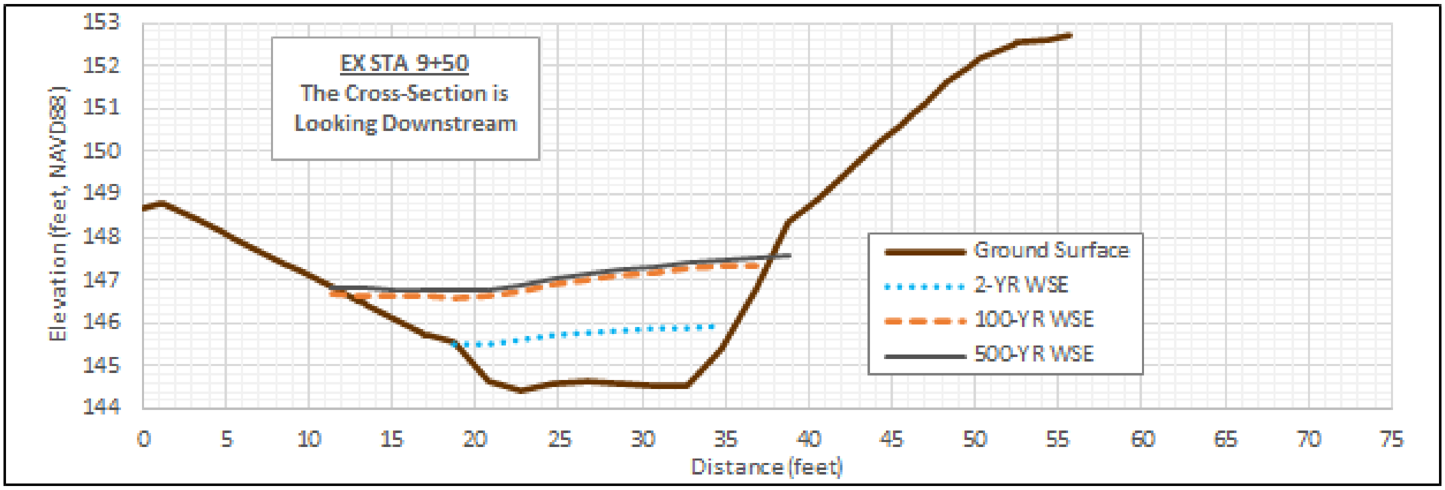


Fig 17. Natural Downstream Cross-Section E PR 9+60

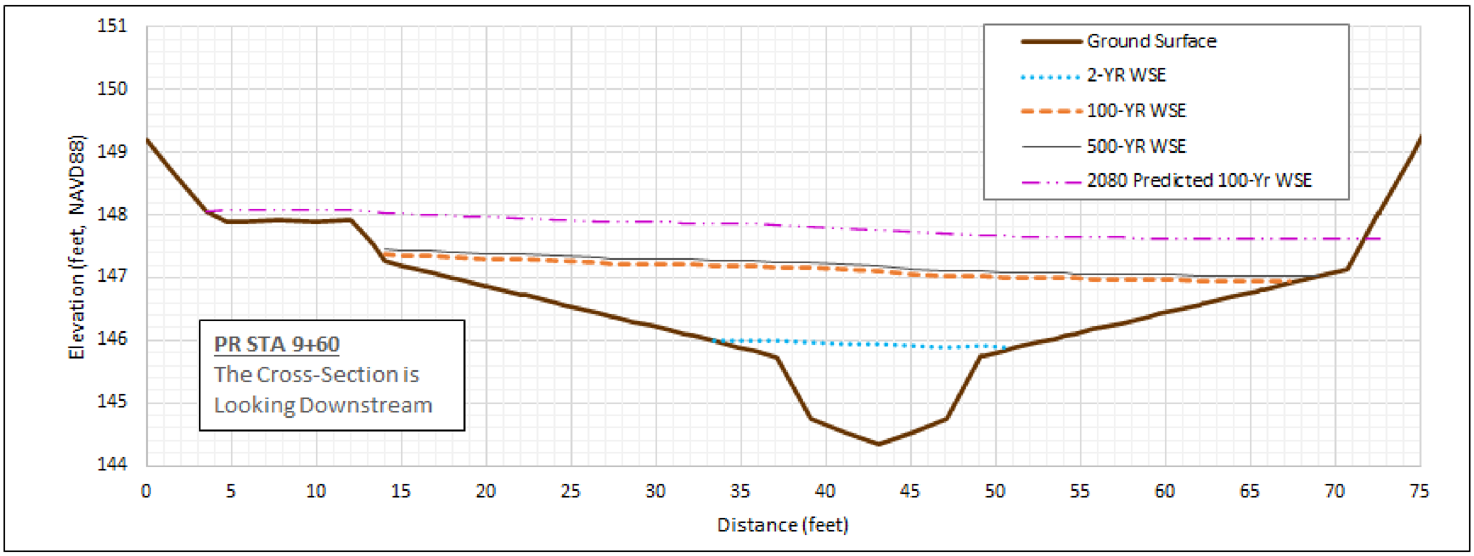


Fig 18. Proposed Downstream Cross-Section E PR 9+60

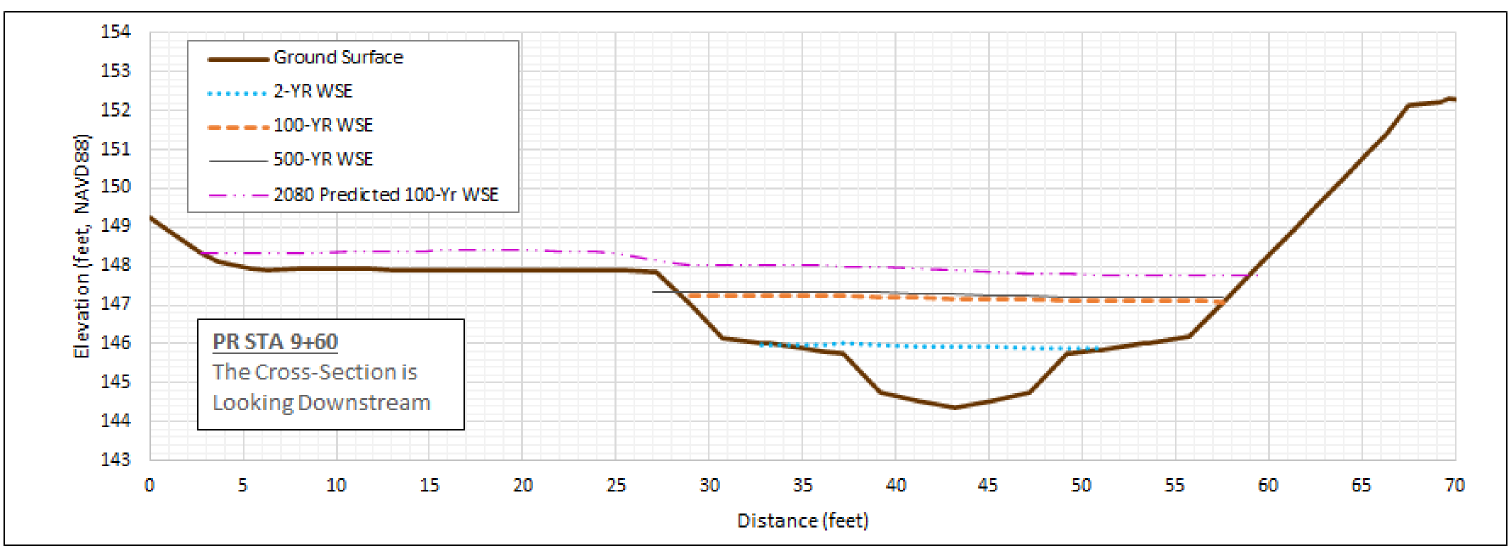


Fig 19. Existing Downstream Cross-Section F EX 11+20

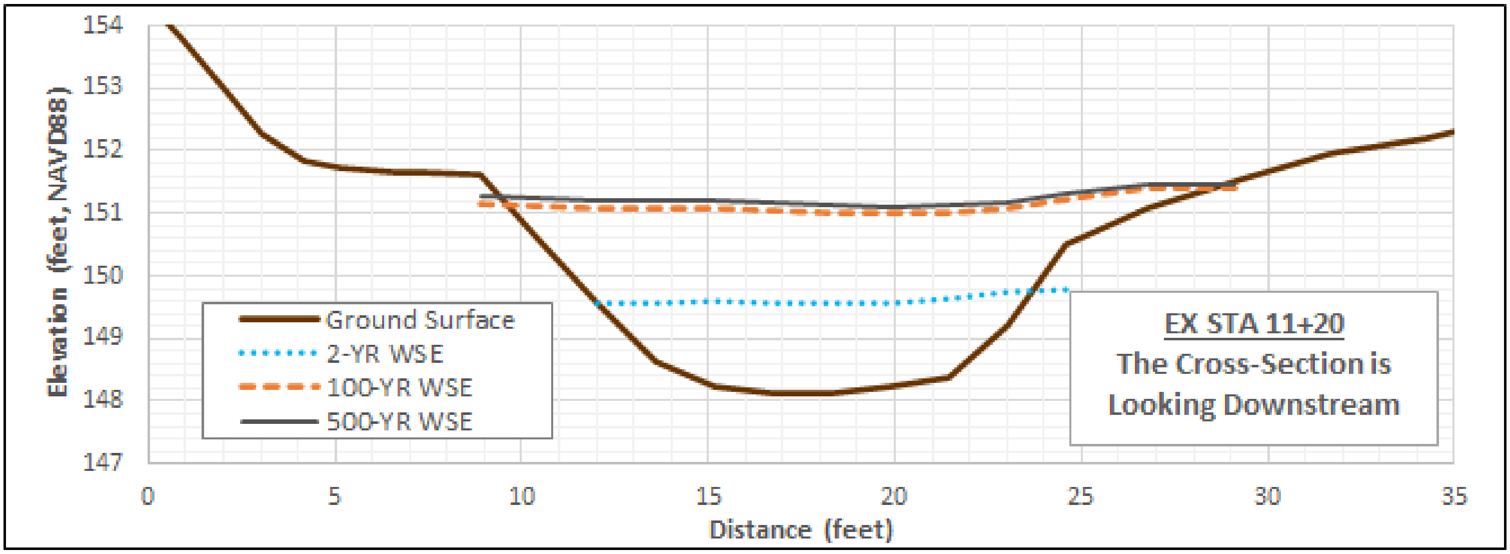


Fig 20. Natural Downstream Cross-Section F PR 10+80

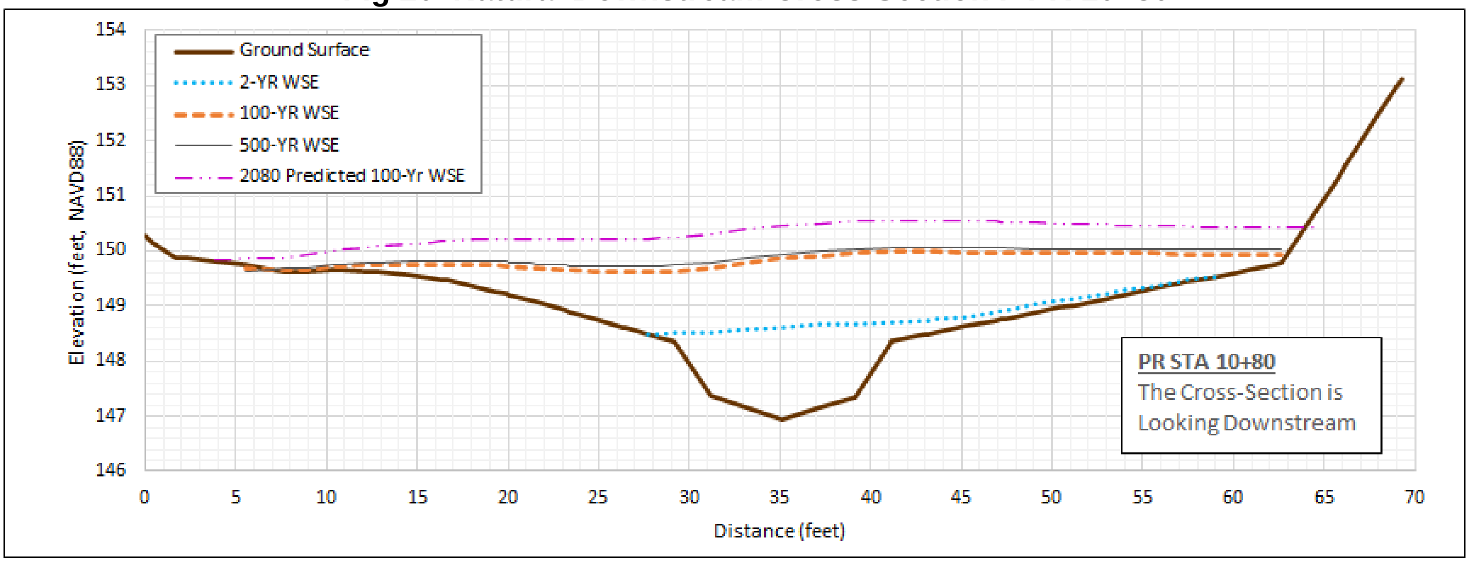


Fig 21. Proposed Downstream Cross-Section F PR 10+80

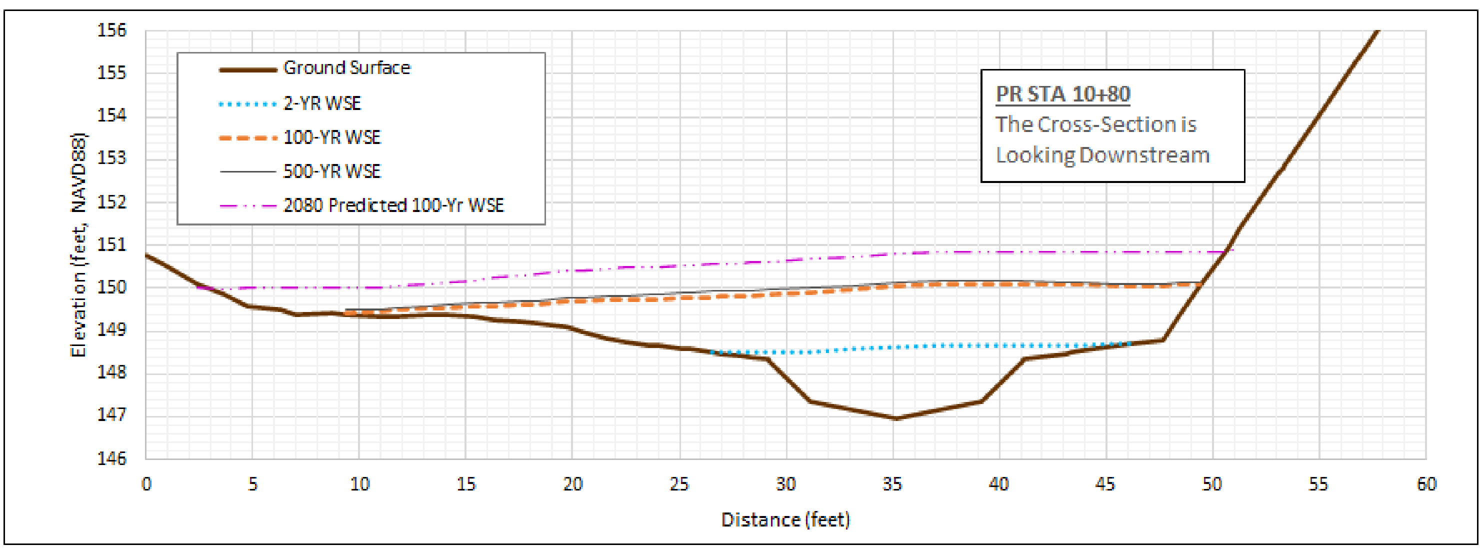


Fig 22. Natural Through Structure Cross-Section G PR 11+70

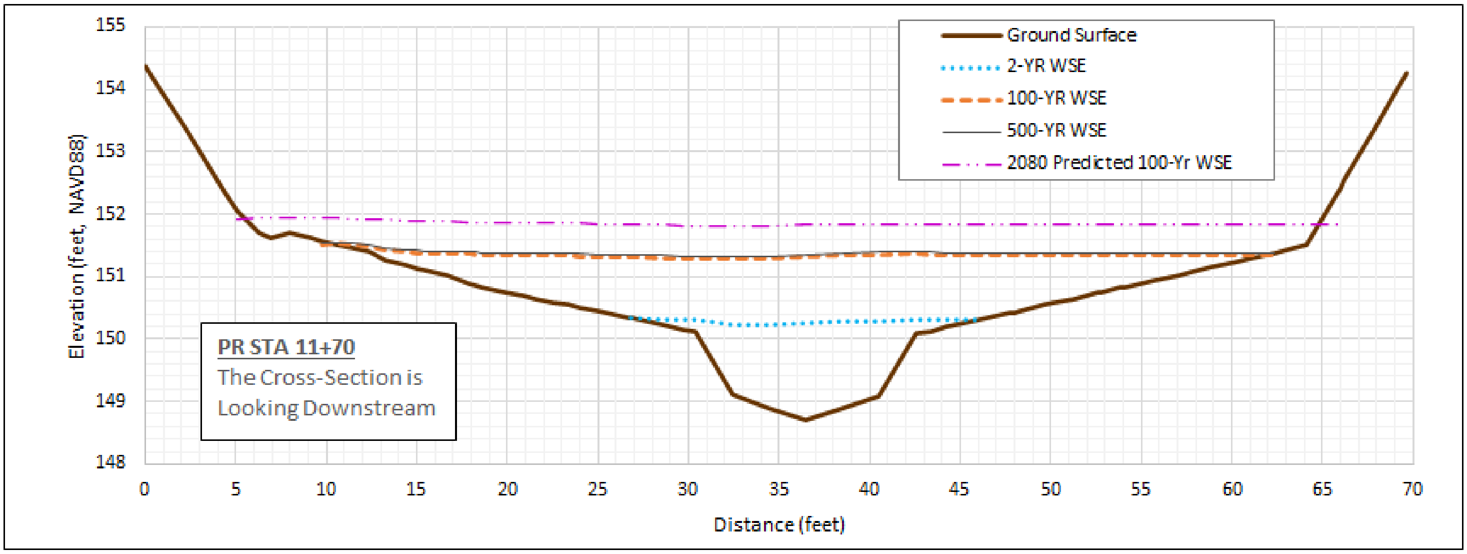


Fig 23. Proposed Through Structure Cross-Section G PR 11+70

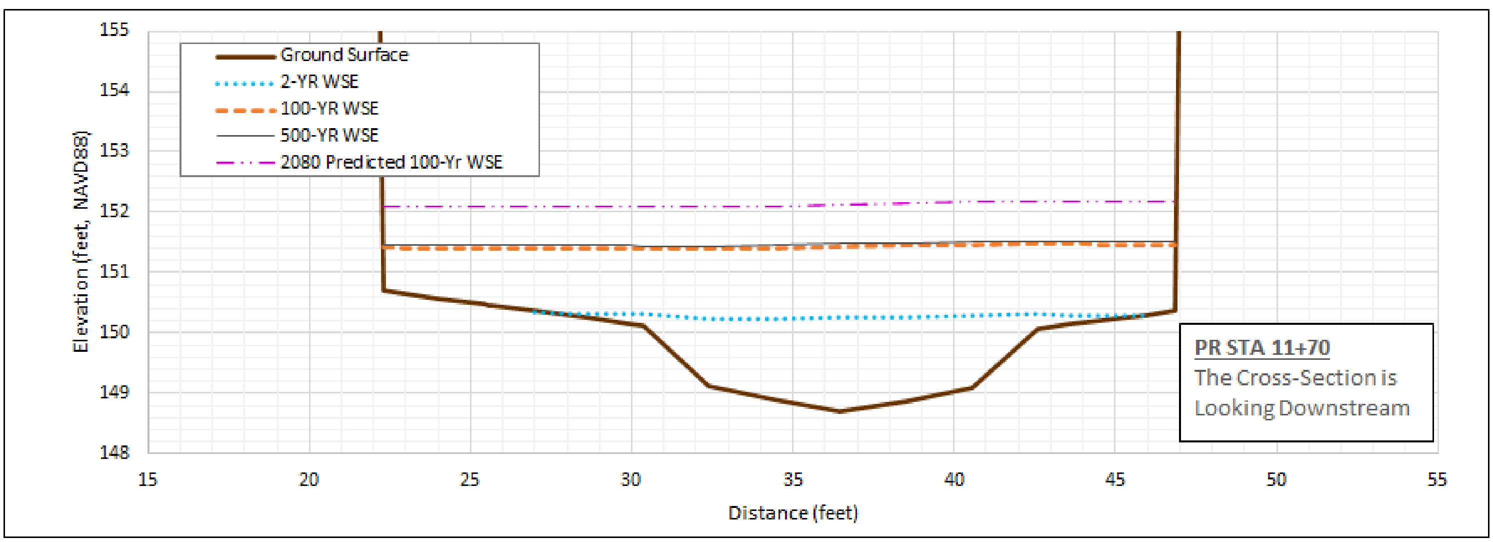


Fig 24. Existing Upstream Cross-Section H EX 12+90

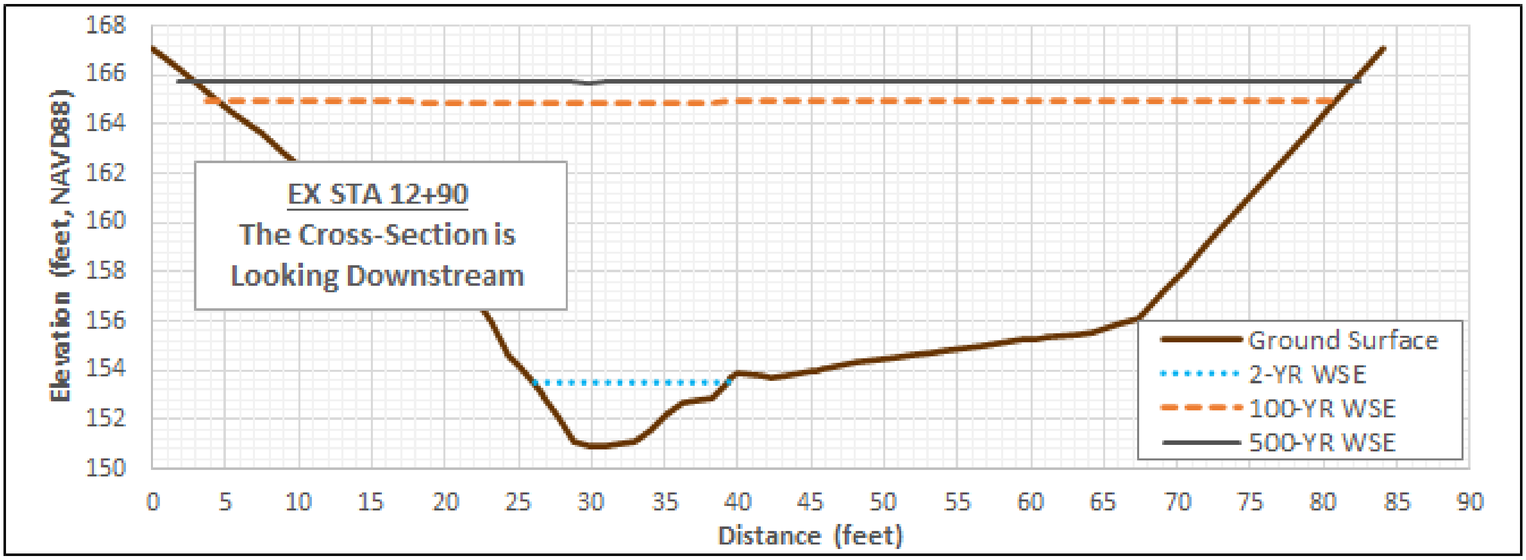


Fig 25. Natural Upstream Cross-Section H PR 12+40

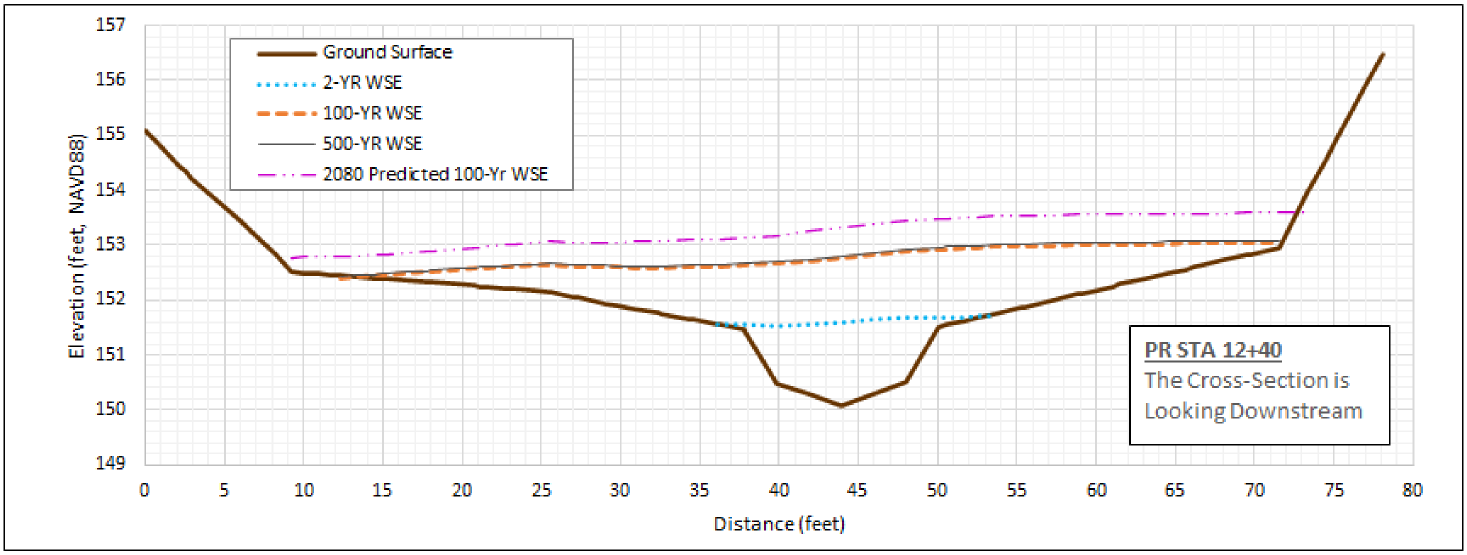


Fig 26. Proposed Upstream Cross-Section H PR 12+40

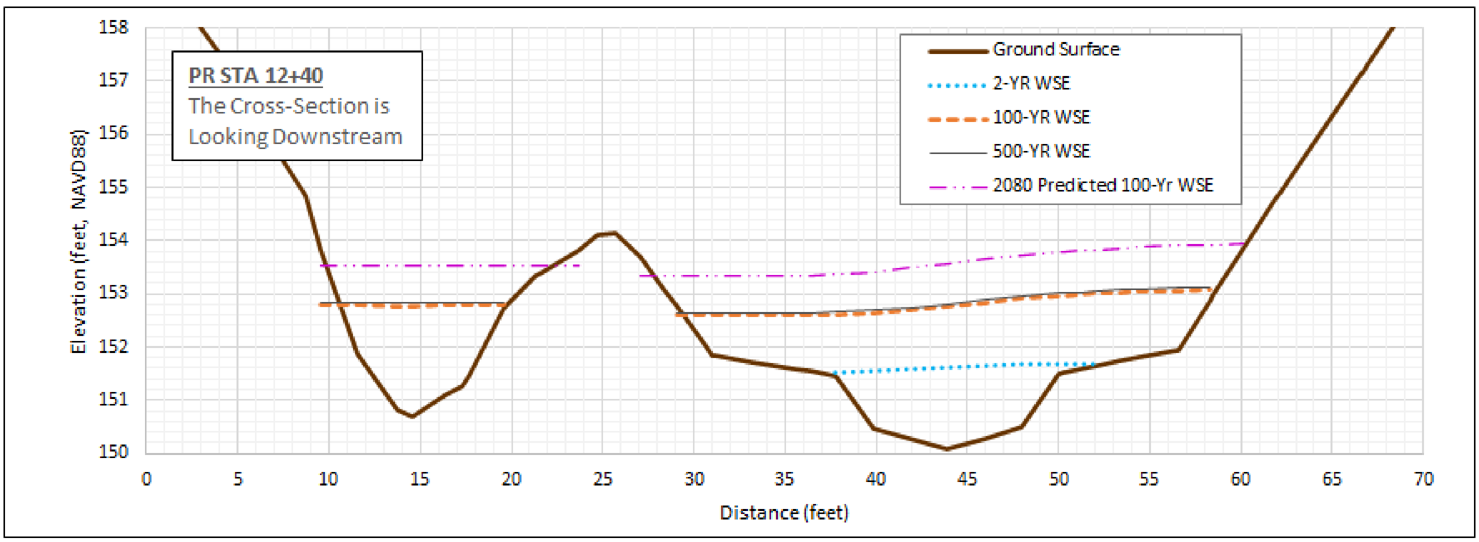


Fig 27. Existing Upstream Cross-Section I EX 14+00

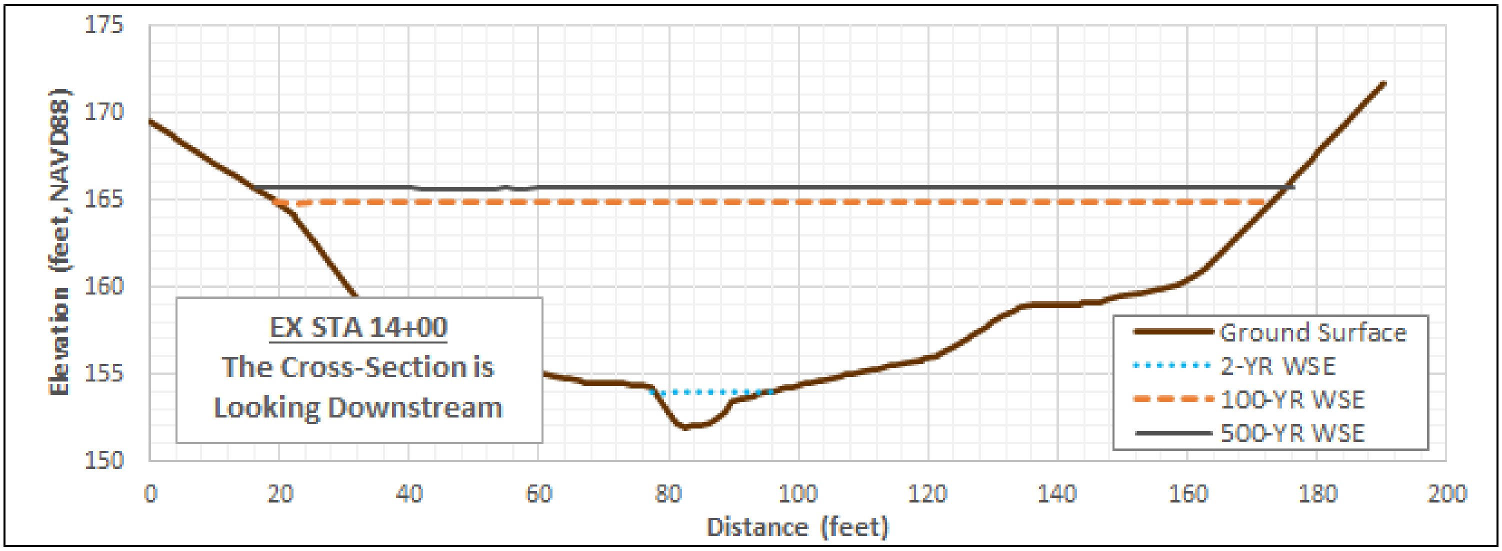


Fig 28. Natural Upstream Cross-Section I PR 13+40

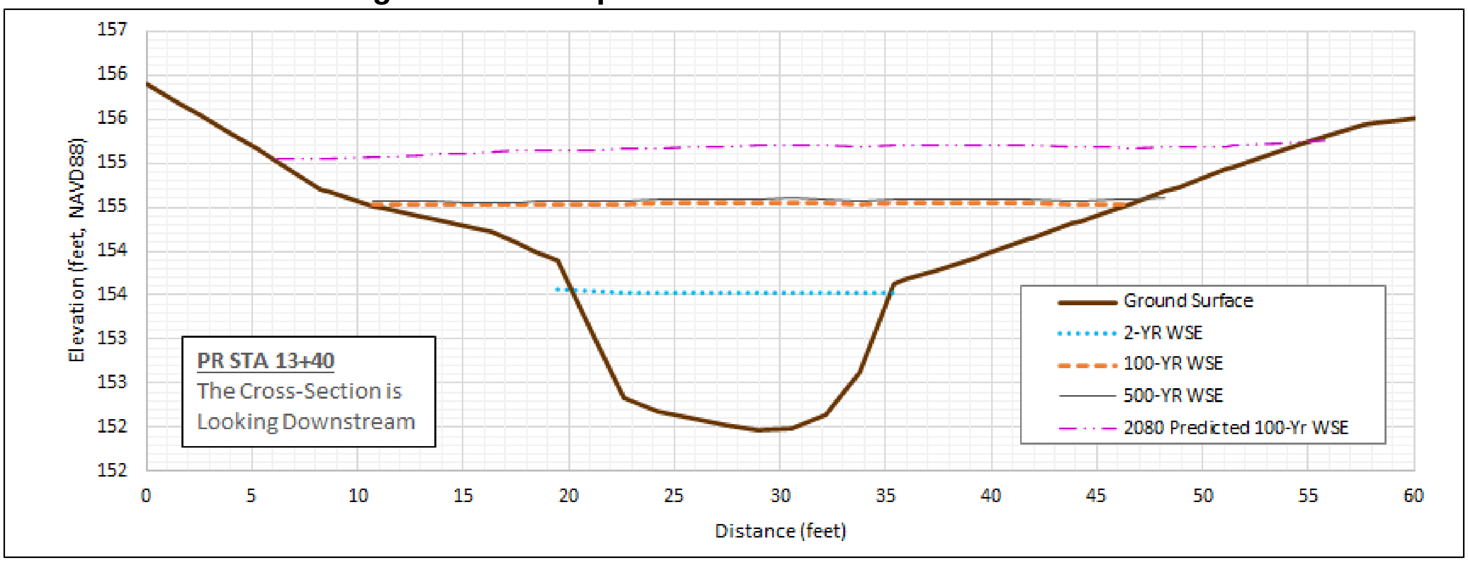


Fig 29. Proposed Upstream Cross-Section I PR 13+40

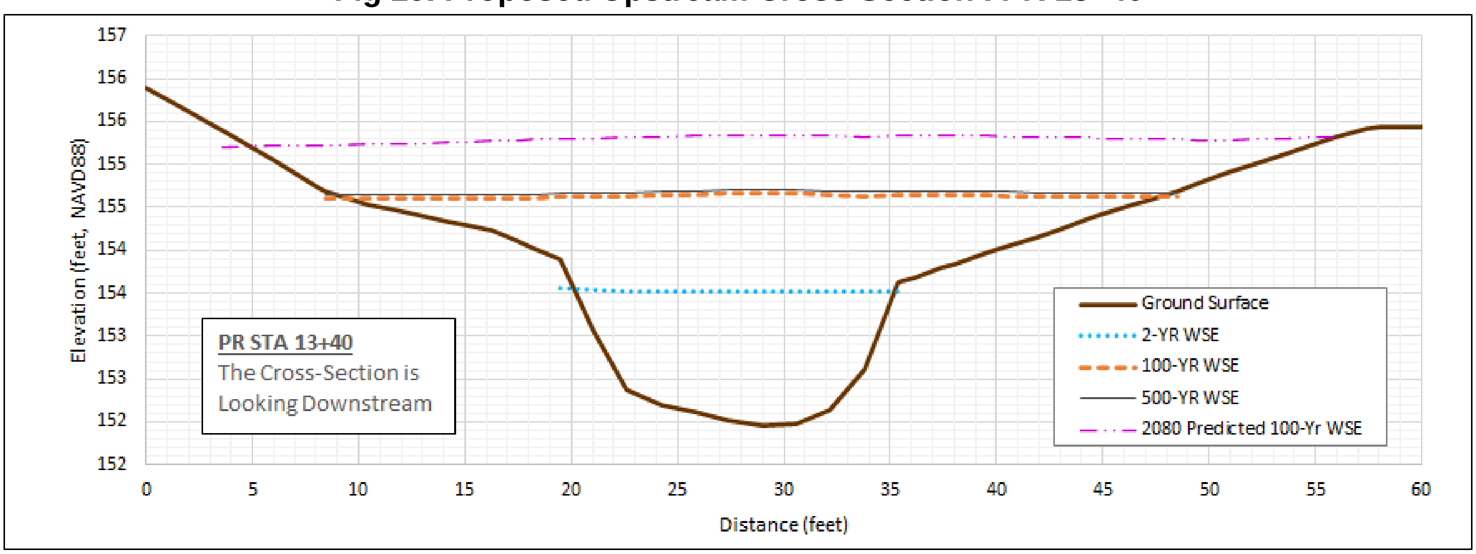


Fig 30. Existing Upstream Cross-Section J EX 14+40

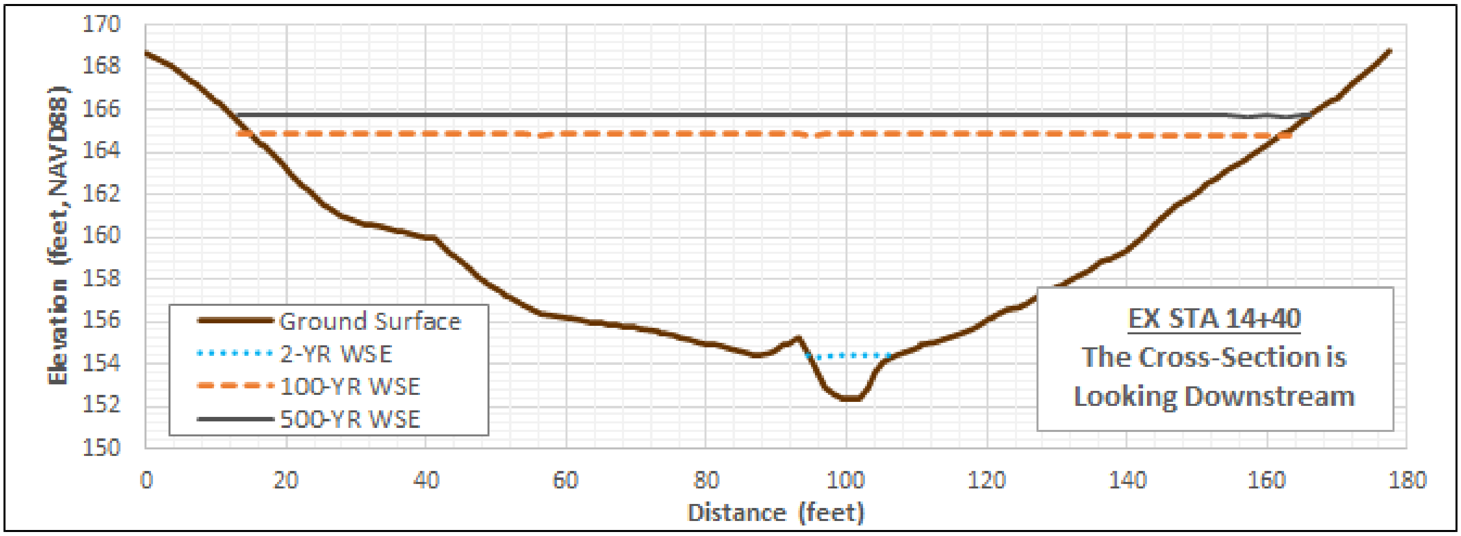


Fig 31. Natural Upstream Cross-Section J PR 14+00

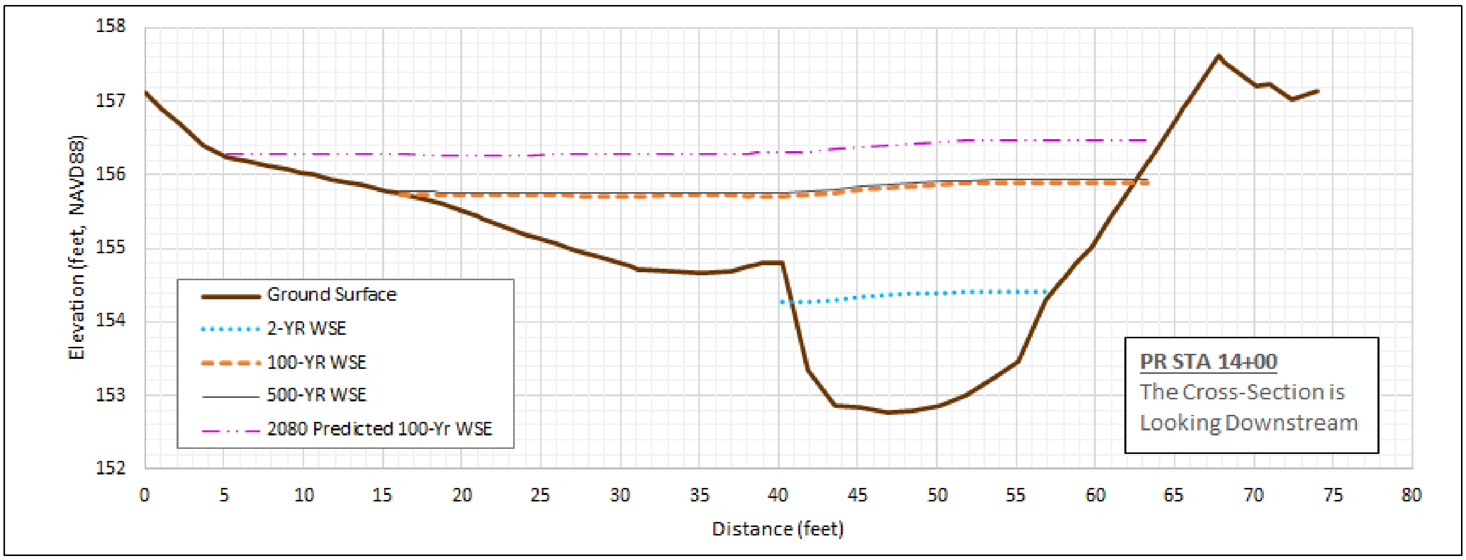
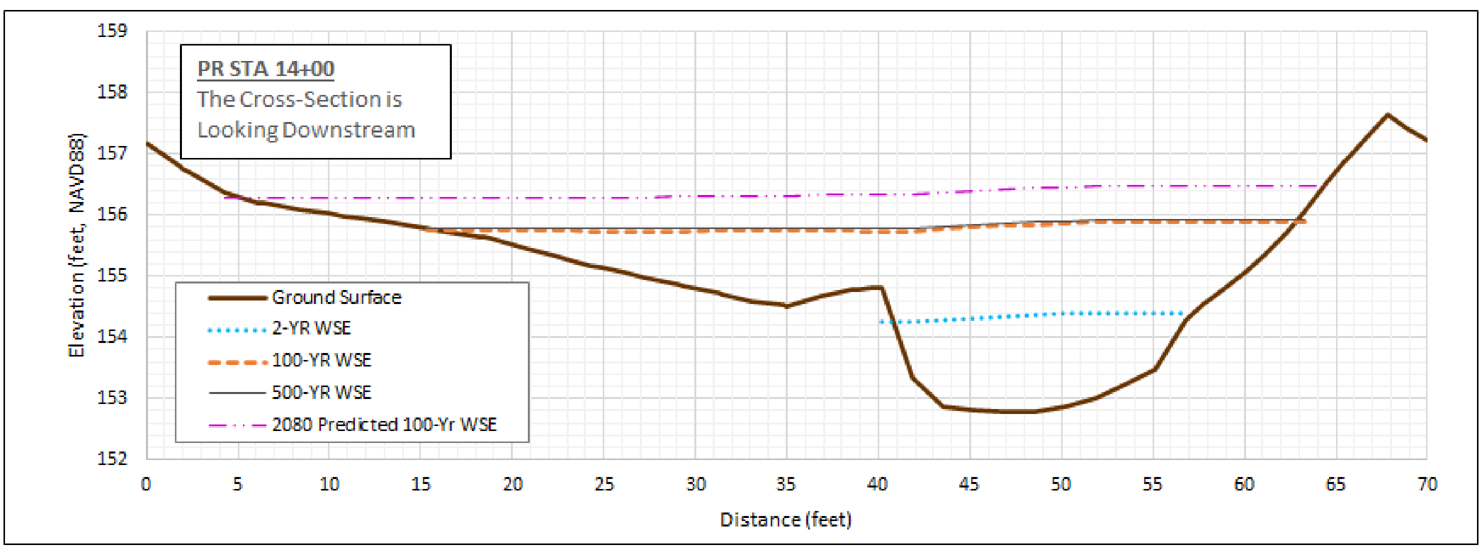


Fig 32. Proposed Upstream Cross-Section J PR 14+00



WATER SURFACE ELEVATION

EXISTING CONDITION 2-YEAR EVENT

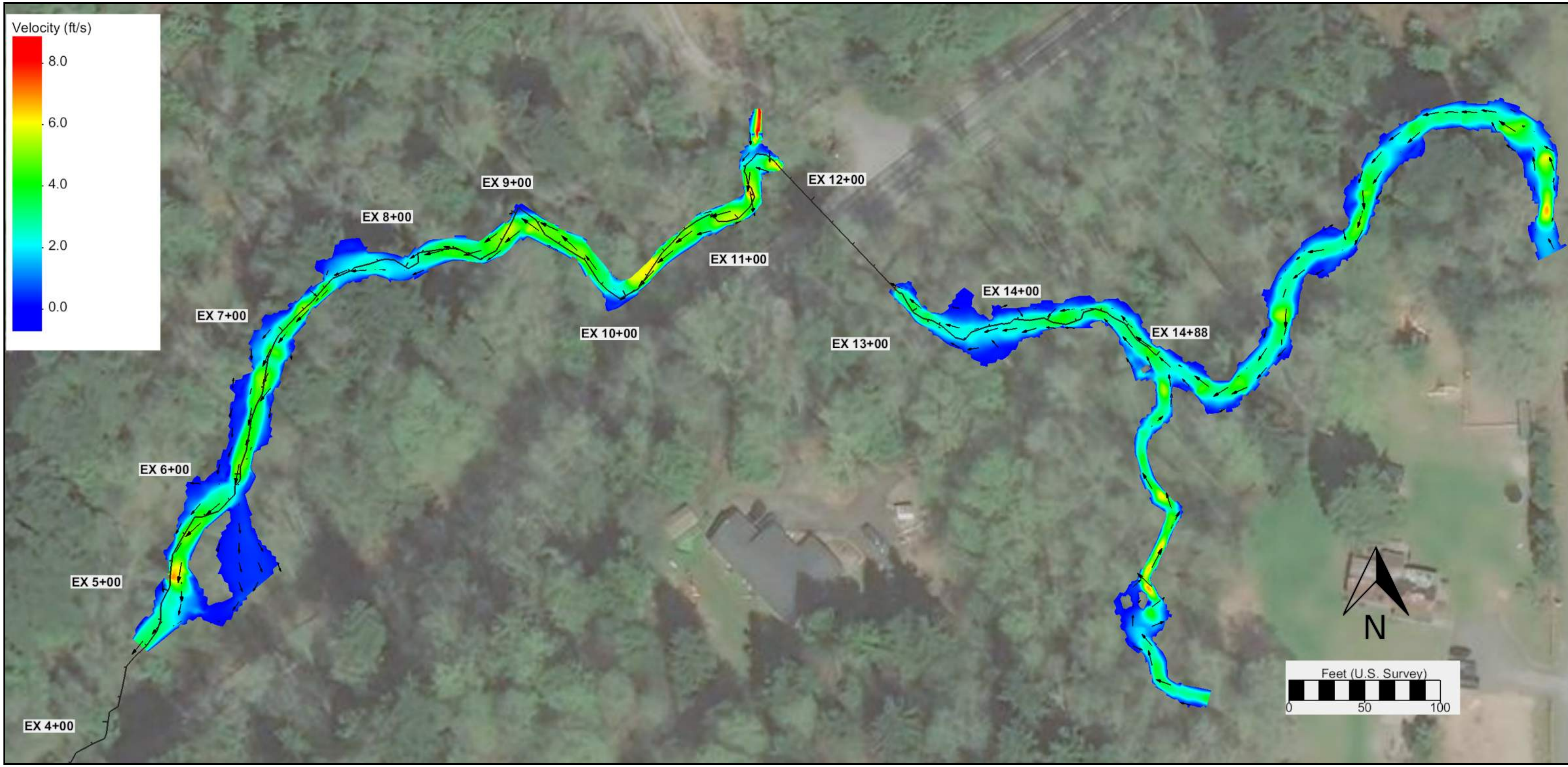
SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PACE An Engineering Services Company

Washington State Department of Transportation

Depth (ft)
4.0
3.0
2.0
1.0
0.0
EX 5+00
EX 6+00
EX 7+00
EX 8+00
EX 9+00
EX 10+00
EX 11+00
EX 12+00
EX 13+00
EX 14+00
EX 14+88
+00
N
Feet (U.S. Survey)
0
50
100

DEPTH

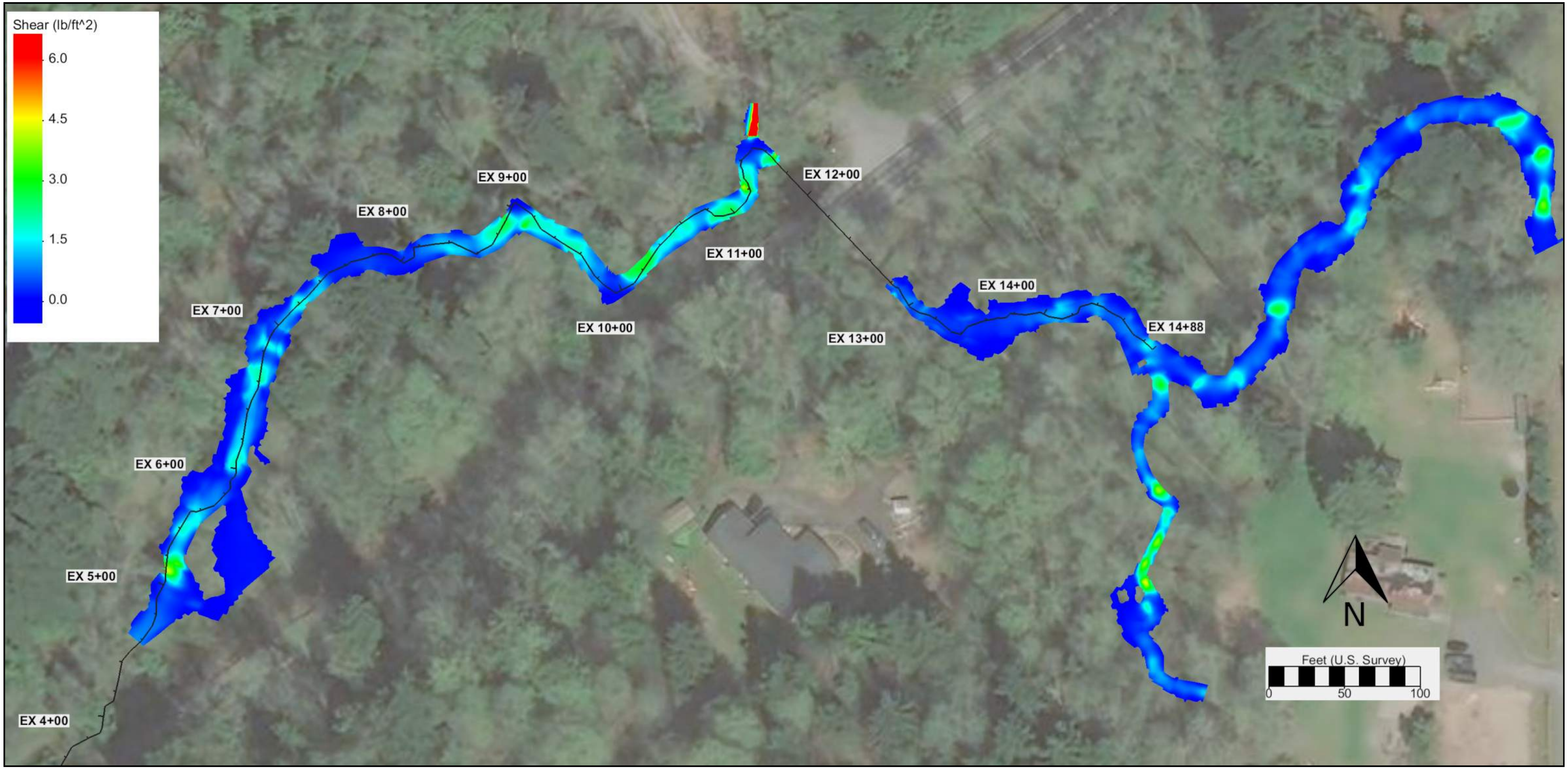


VELOCITY

EXISTING CONDITION 2-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN

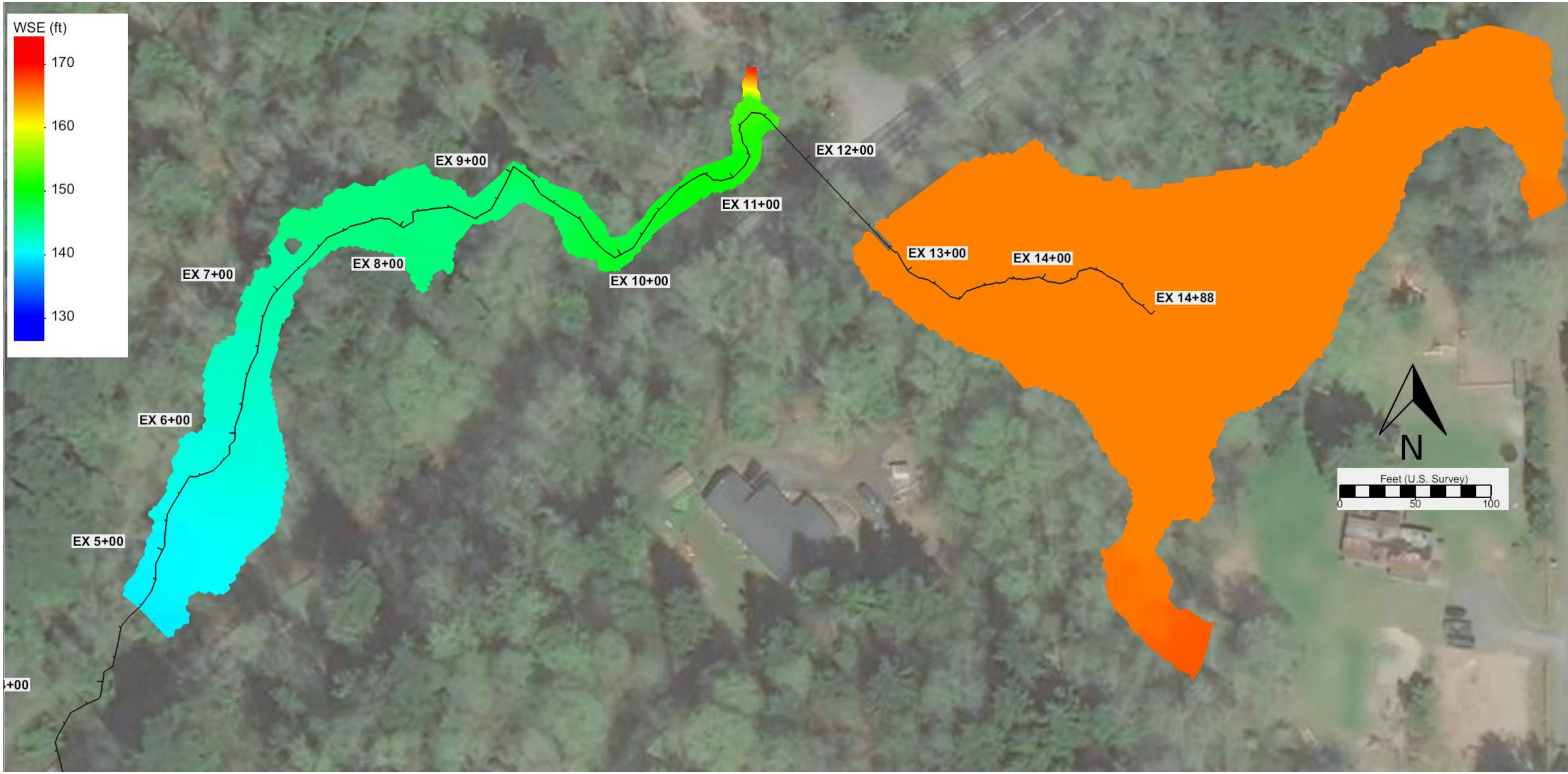


SHEAR

Washington State Department of Transportation

EXISTING CONDITION 2-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK



WATER SURFACE ELEVATION

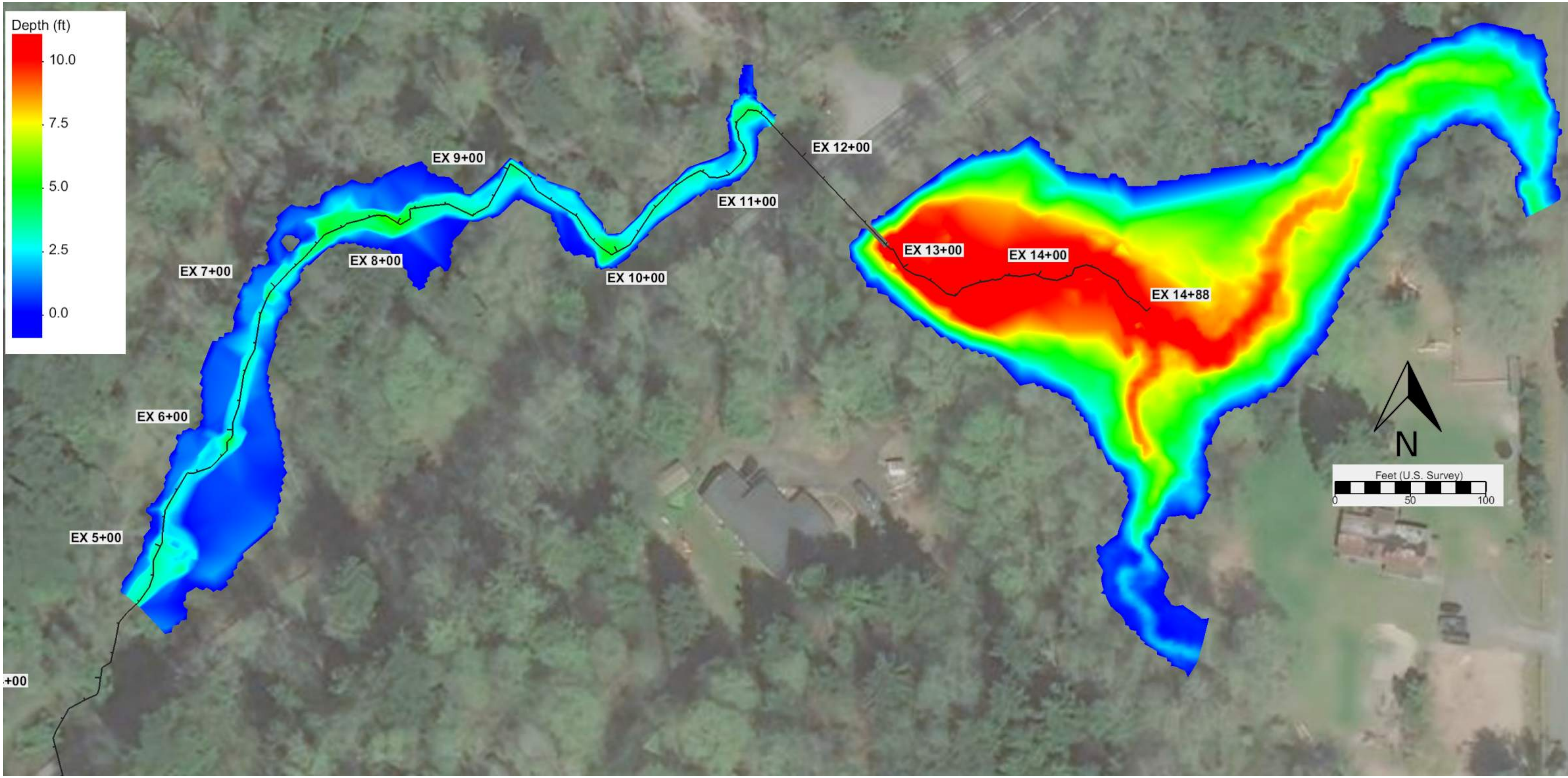
EXISTING CONDITION 100-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

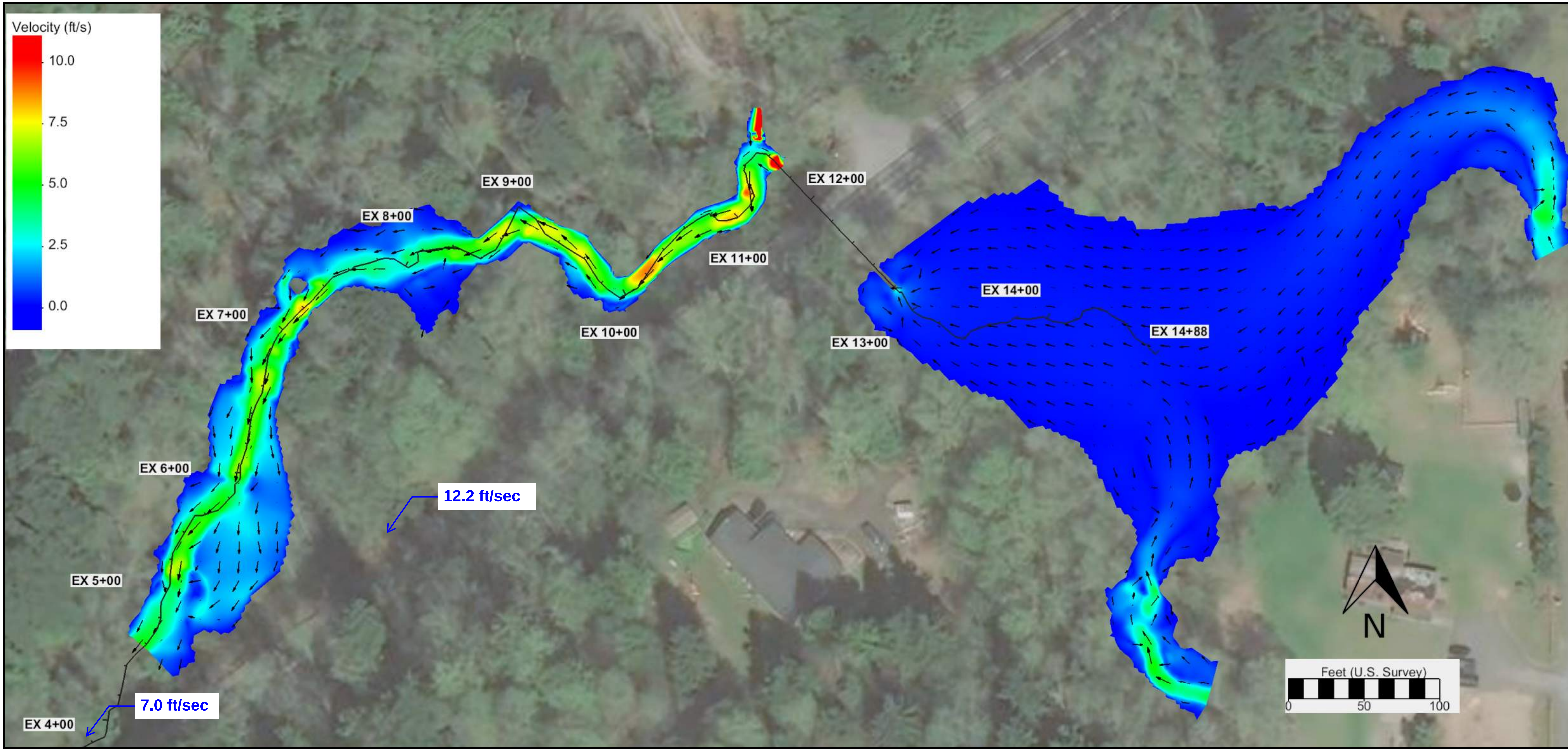
SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



DEPTH

EXISTING CONDITION 100-YEAR EVENT

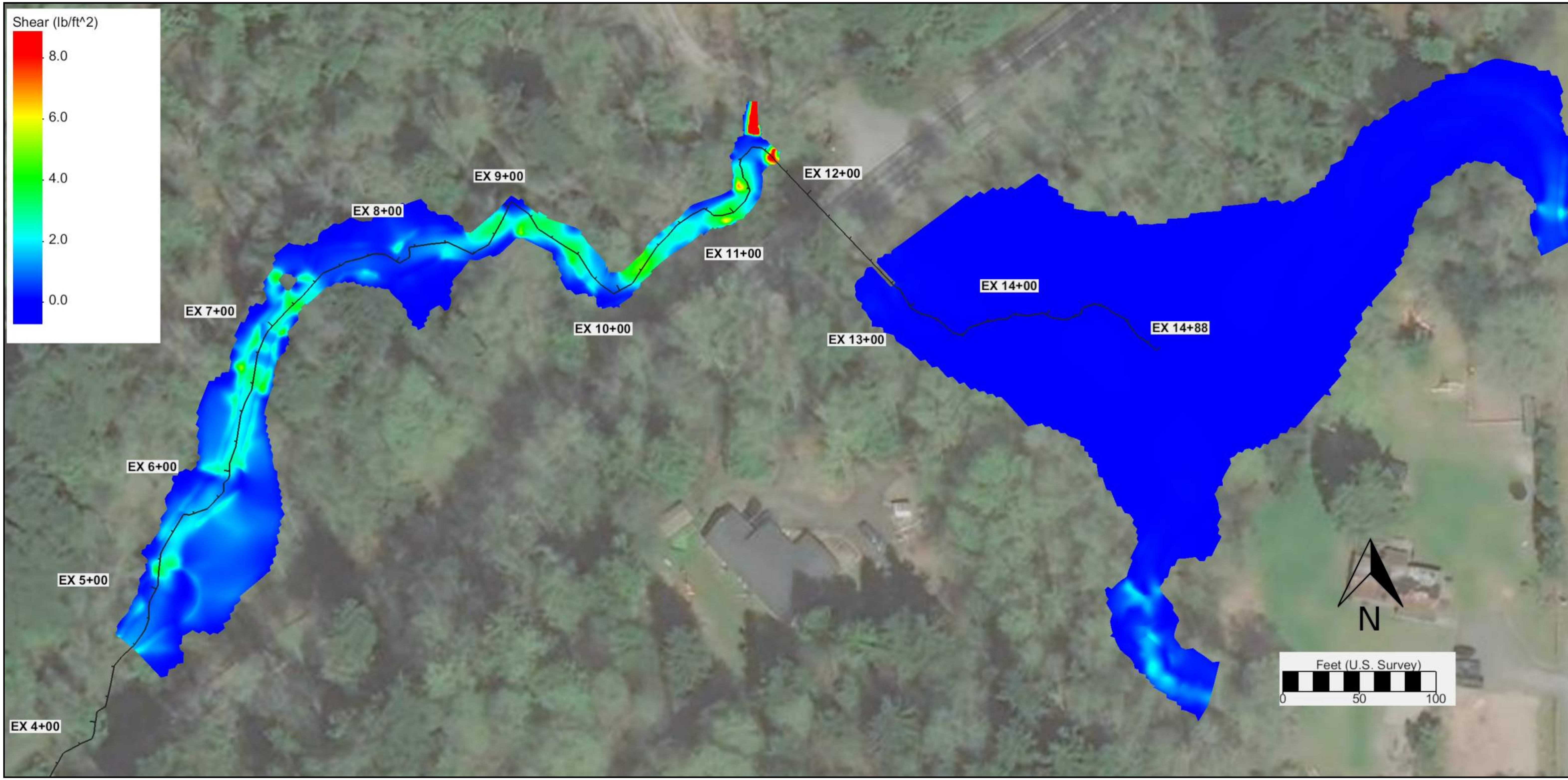


VELOCITY

EXISTING CONDITION 100-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



SHEAR

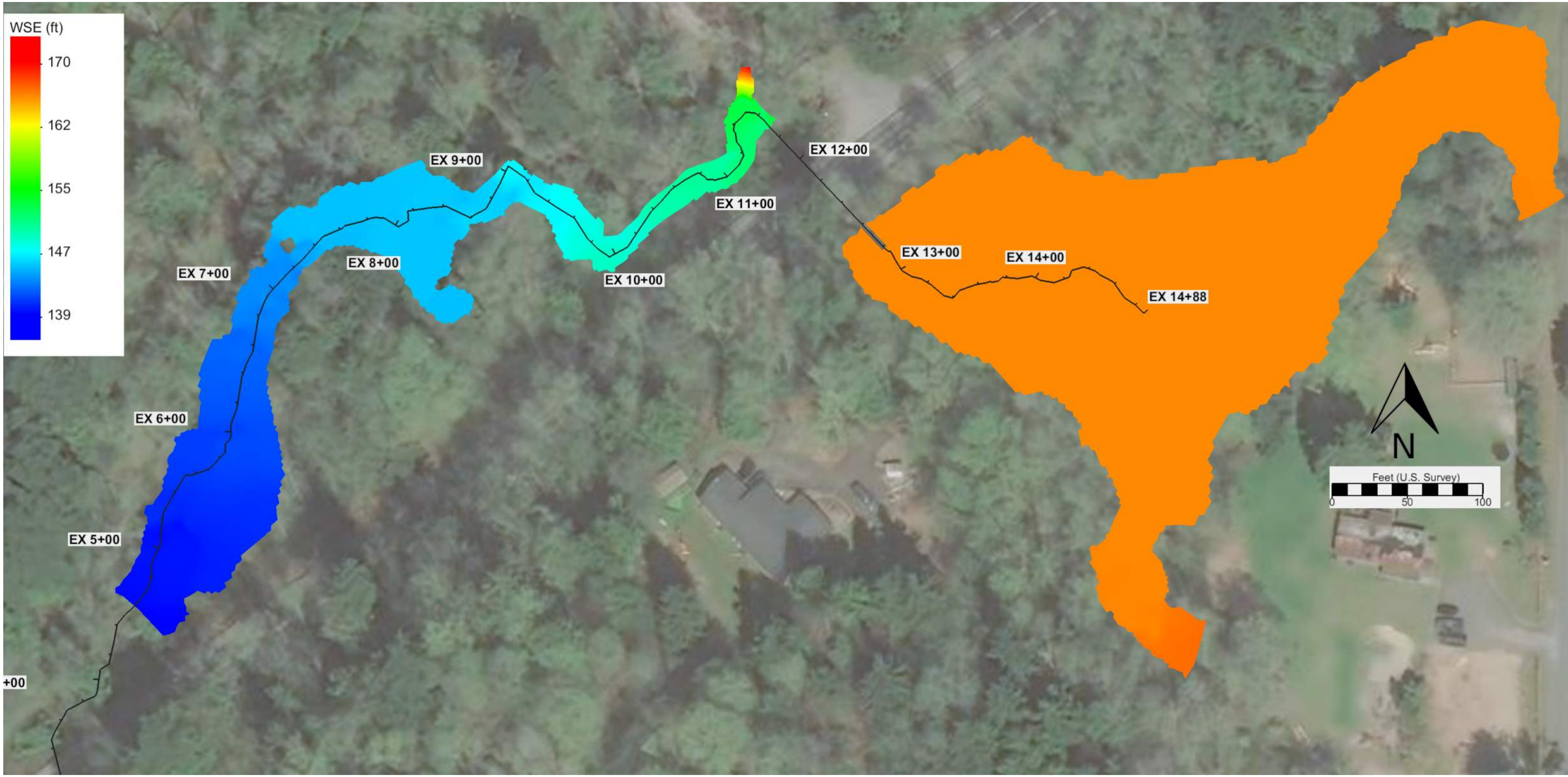
EXISTING CONDITION 100-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



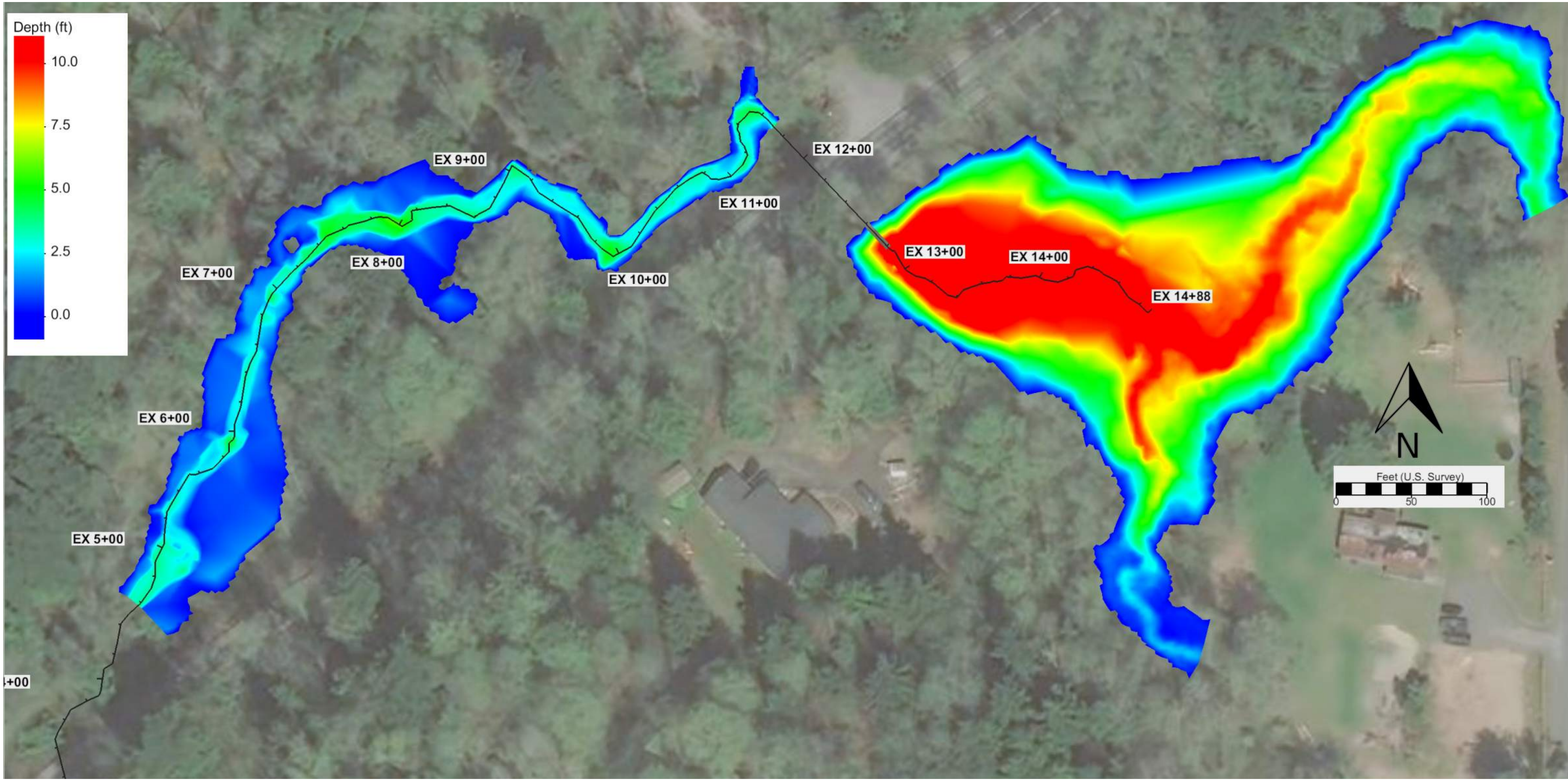
WATER SURFACE ELEVATION

EXISTING CONDITION 500-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PACE An Engineering Services Company

Washington State Department of Transportation



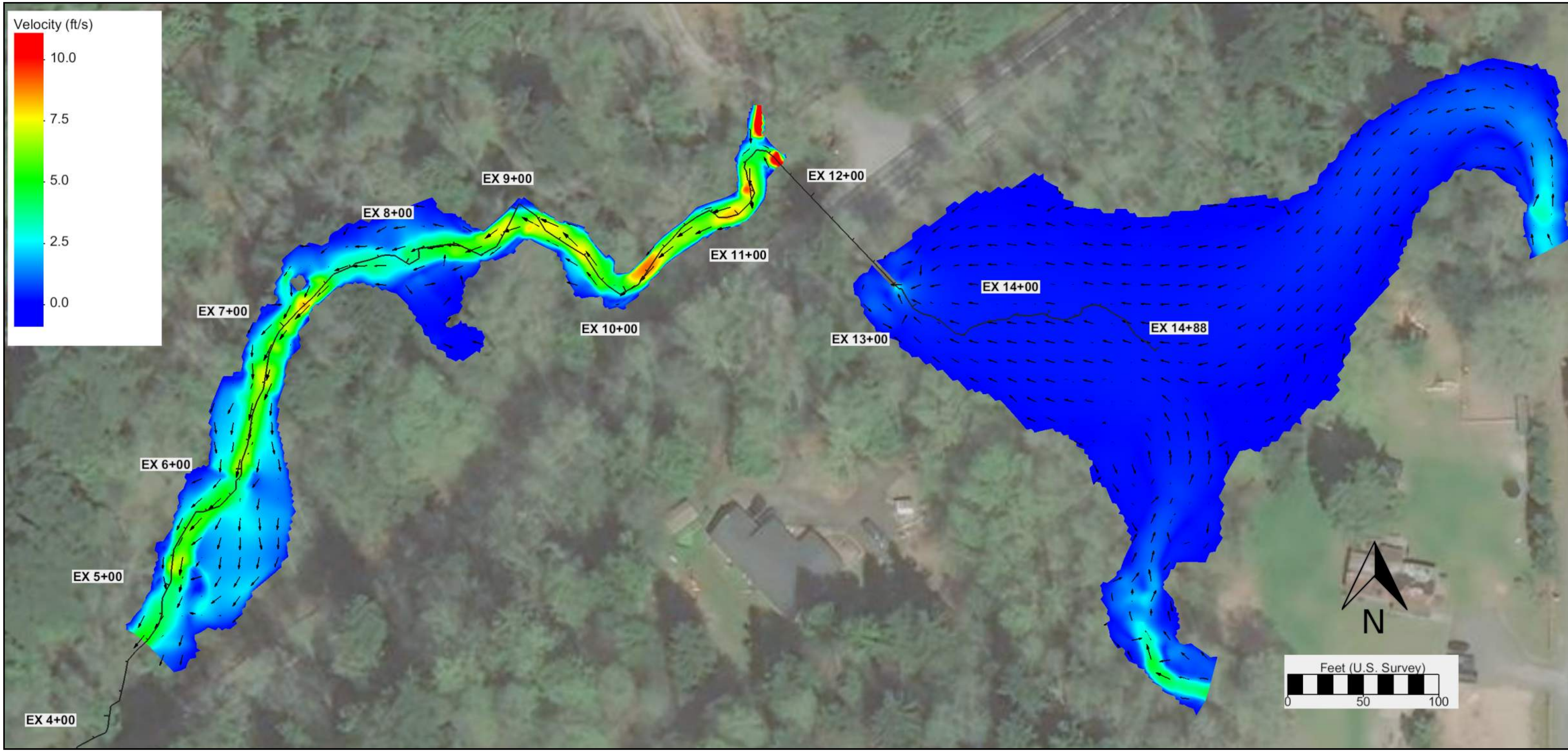
DEPTH

PACE An Engineering Services Company

Washington State Department of Transportation

EXISTING CONDITION 500-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

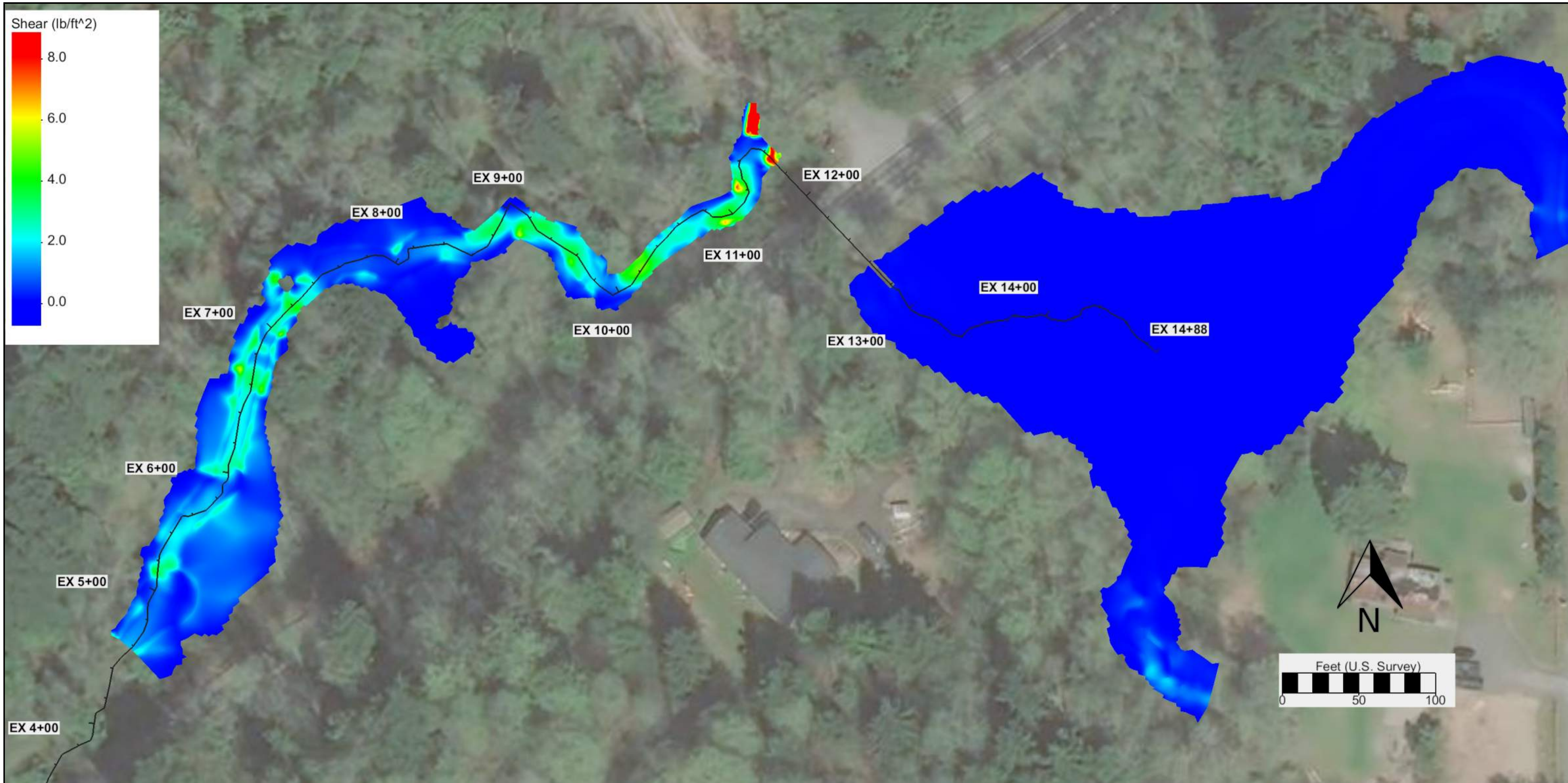


VELOCITY

EXISTING CONDITION 500-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



SHEAR

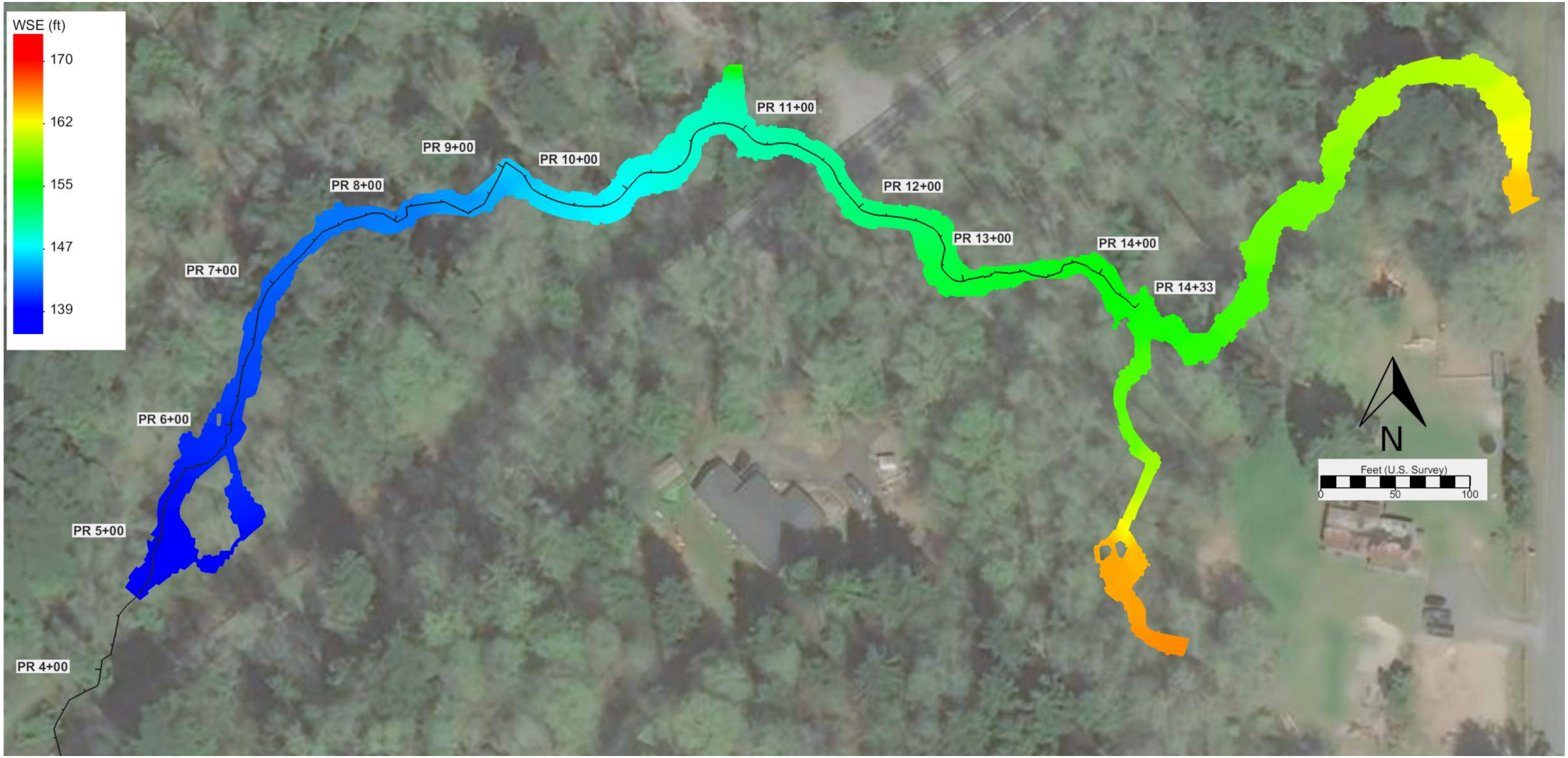
EXISTING CONDITION 500-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



WATER SURFACE ELEVATION

NATURAL CONDITION 2-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

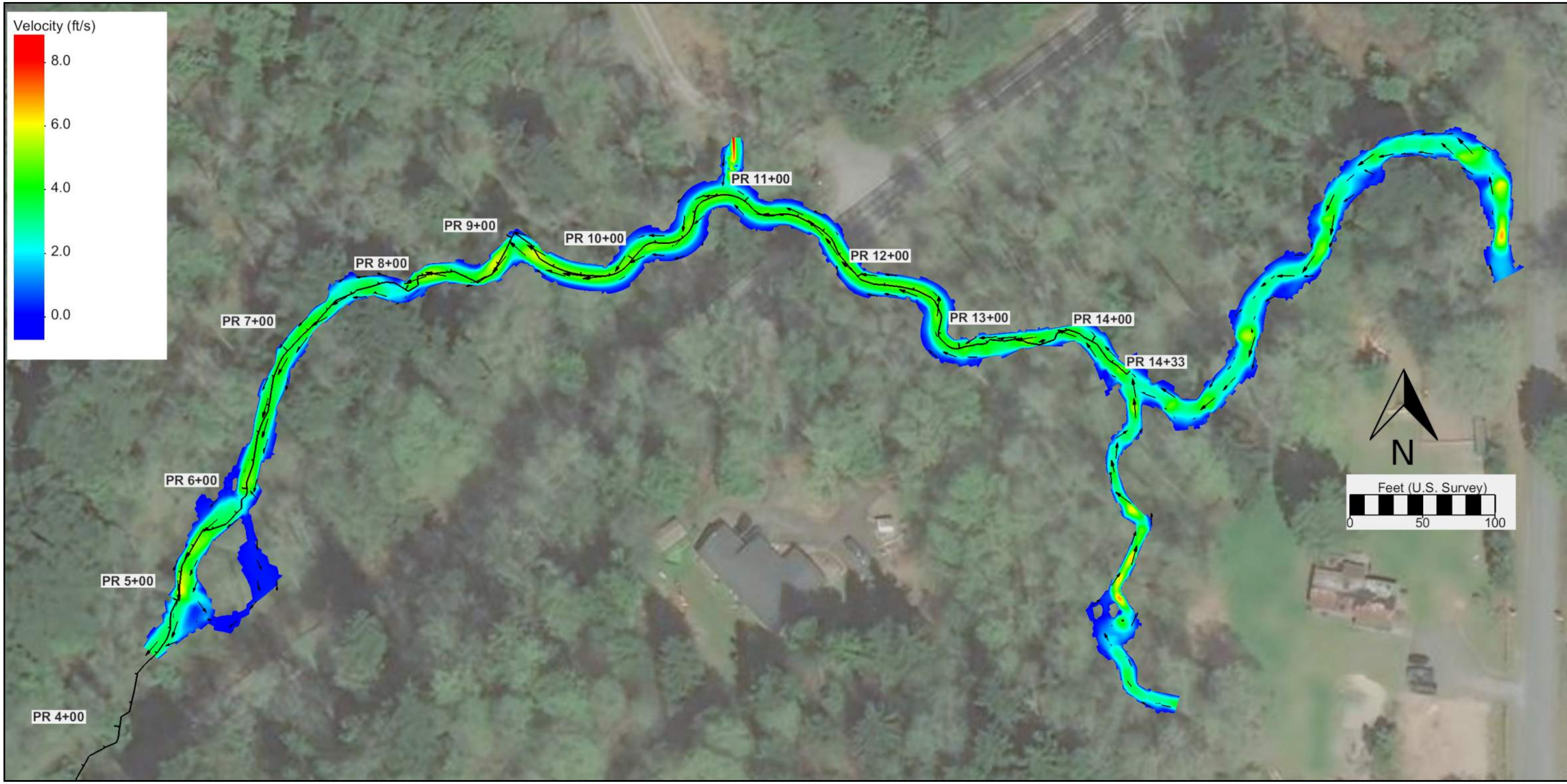
DEPTH

NATURAL CONDITION 2-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK



VELOCITY

NATURAL CONDITION 2-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

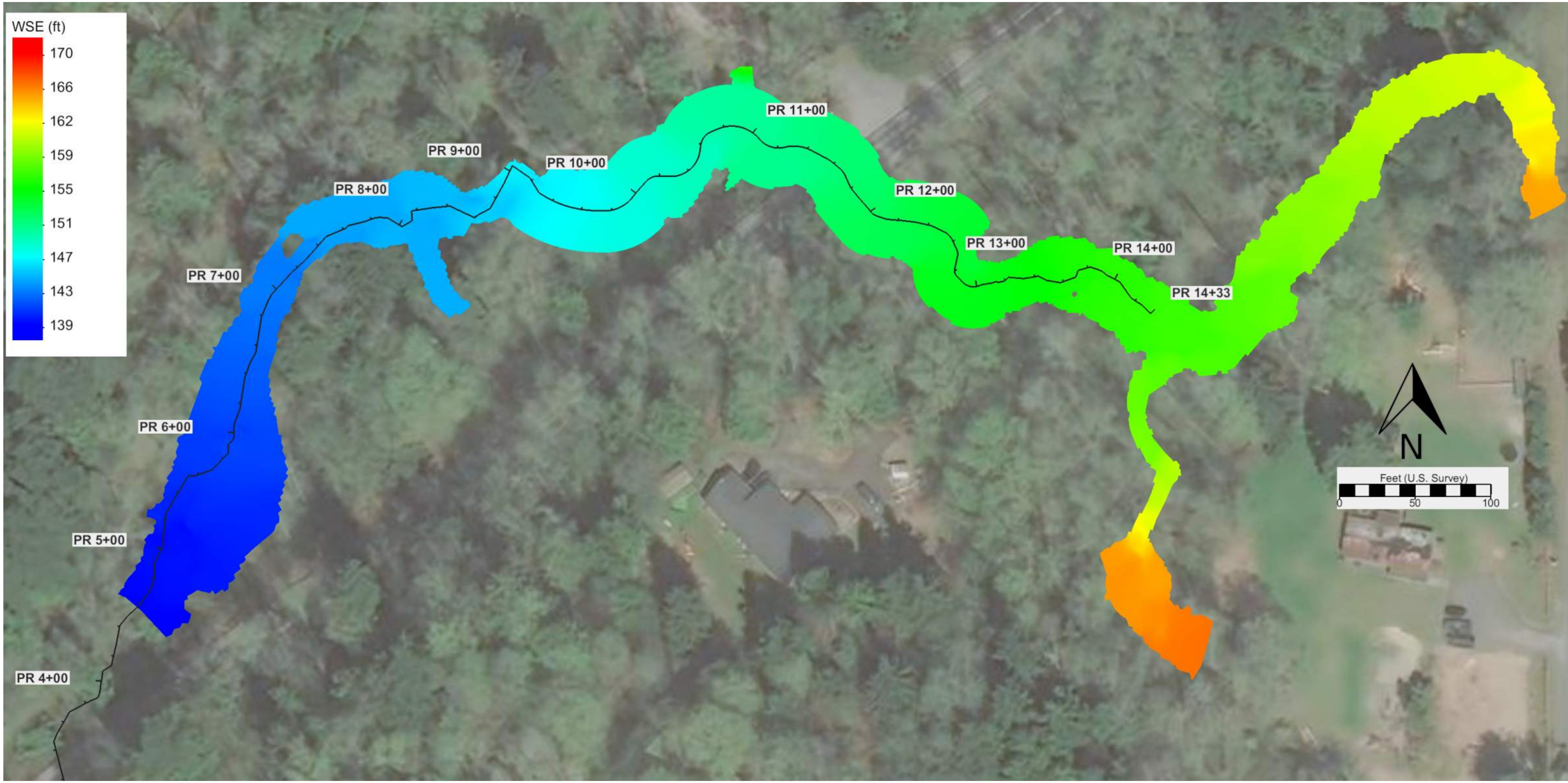
PRELIMINARY HYDRAULIC DESIGN

SHEAR

NATURAL CONDITION 2-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



WATER SURFACE ELEVATION

NATURAL CONDITION 100-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

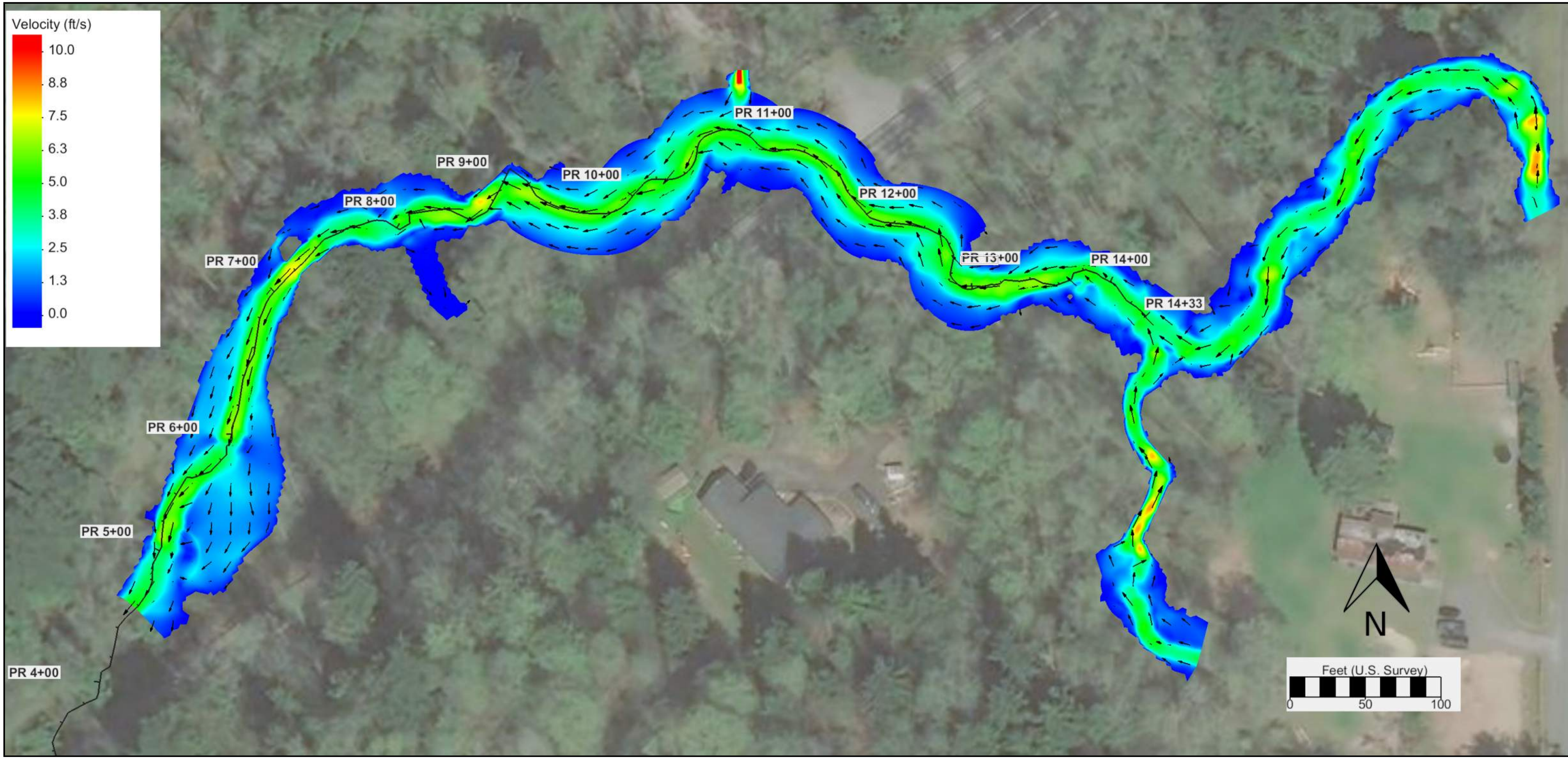
PACE An Engineering Services Company

Washington State Department of Transportation

DEPTH

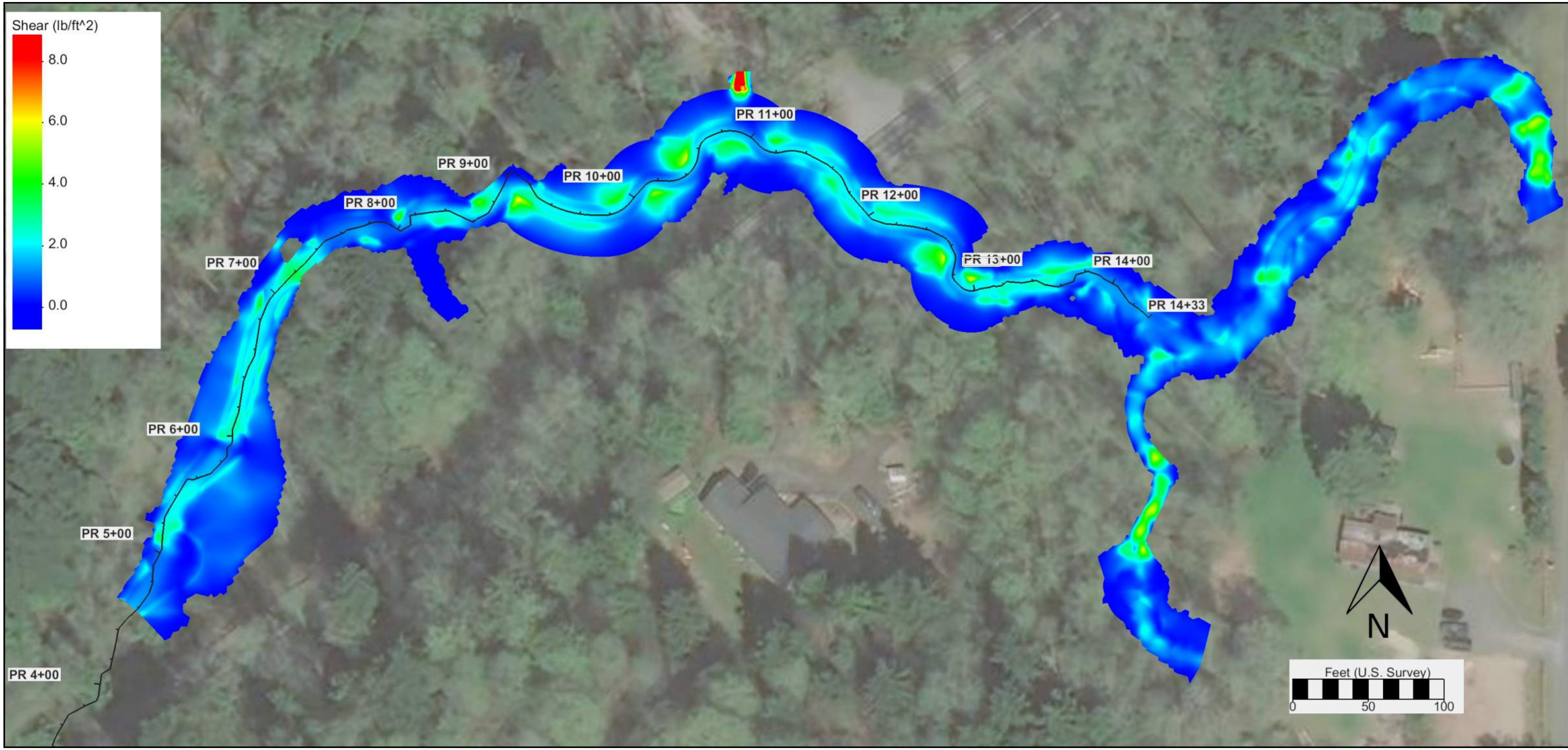
NATURAL CONDITION 100-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK



VELOCITY

NATURAL CONDITION 100-YEAR EVENT



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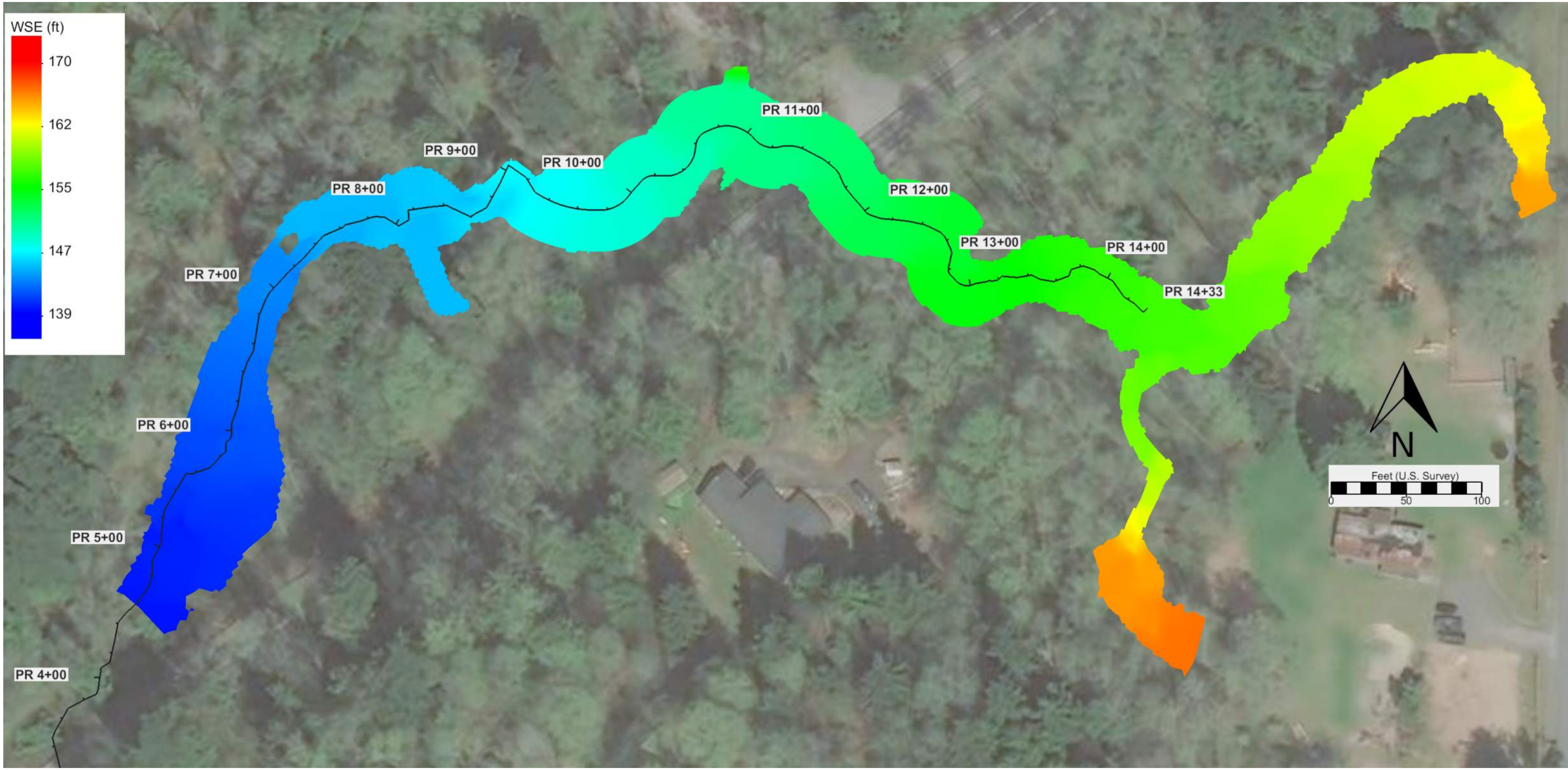
NATURAL CONDITION 100-YEAR EVENT



WATER SURFACE ELEVATION

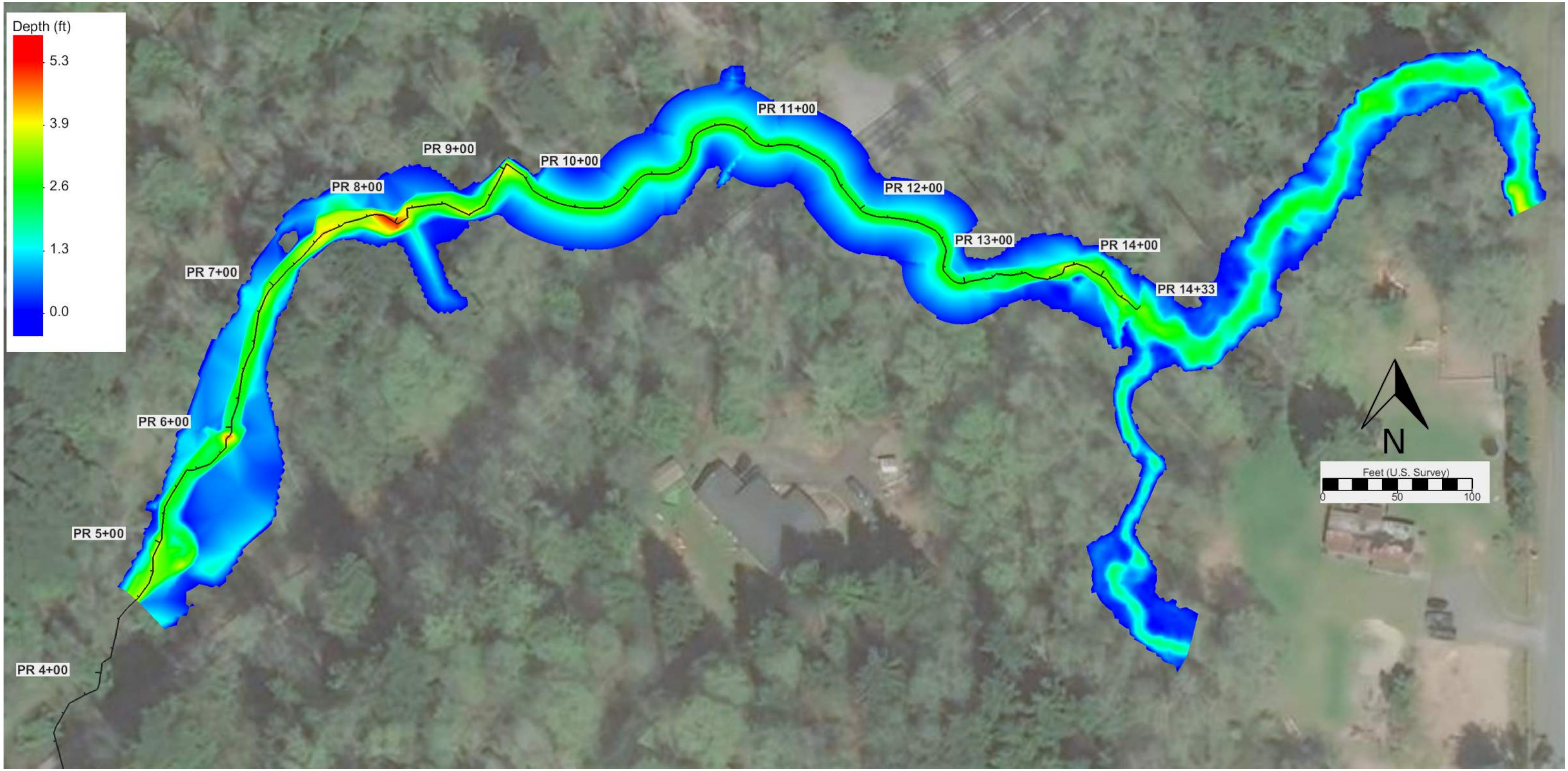
NATURAL CONDITION 500-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



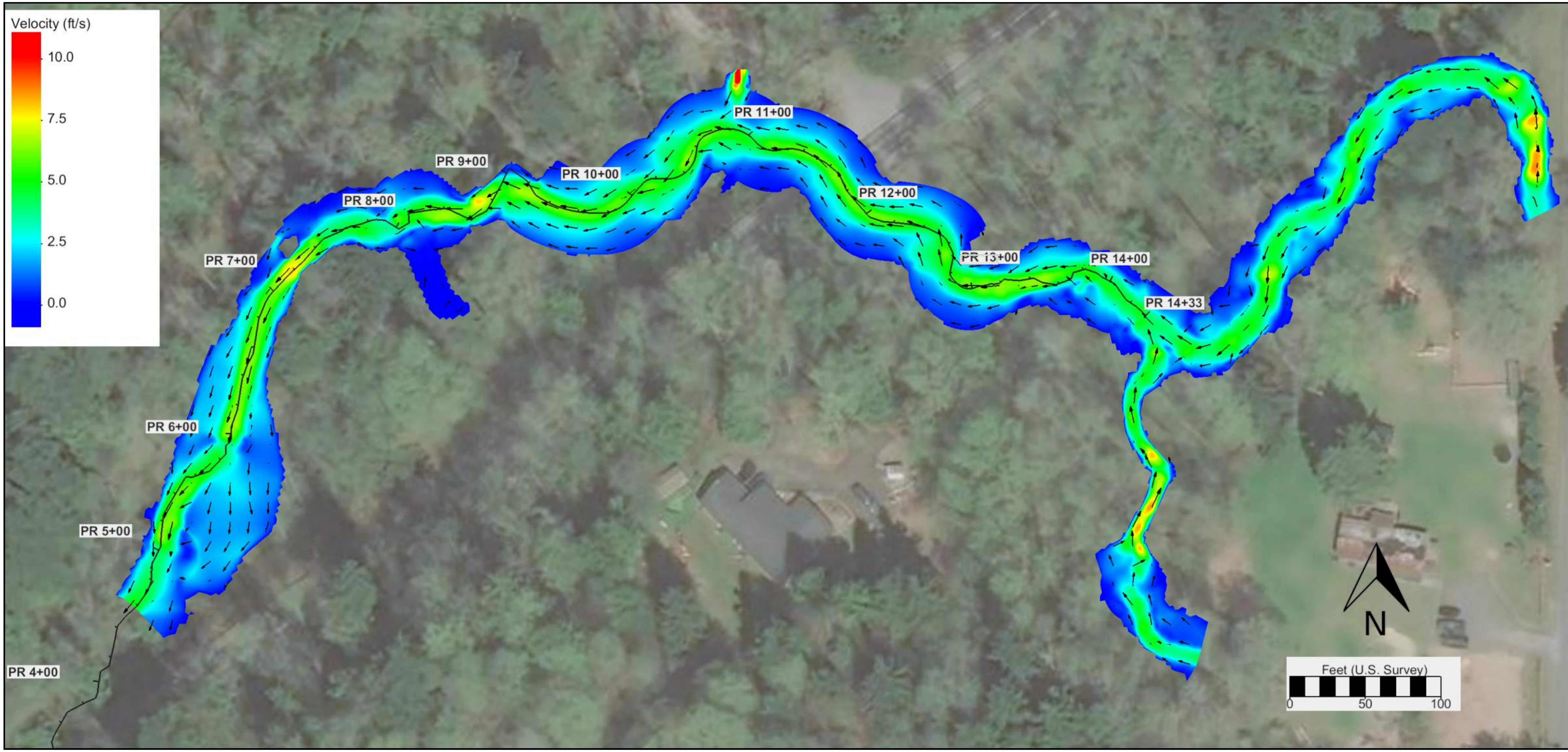
DEPTH

NATURAL CONDITION 500-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK



VELOCITY

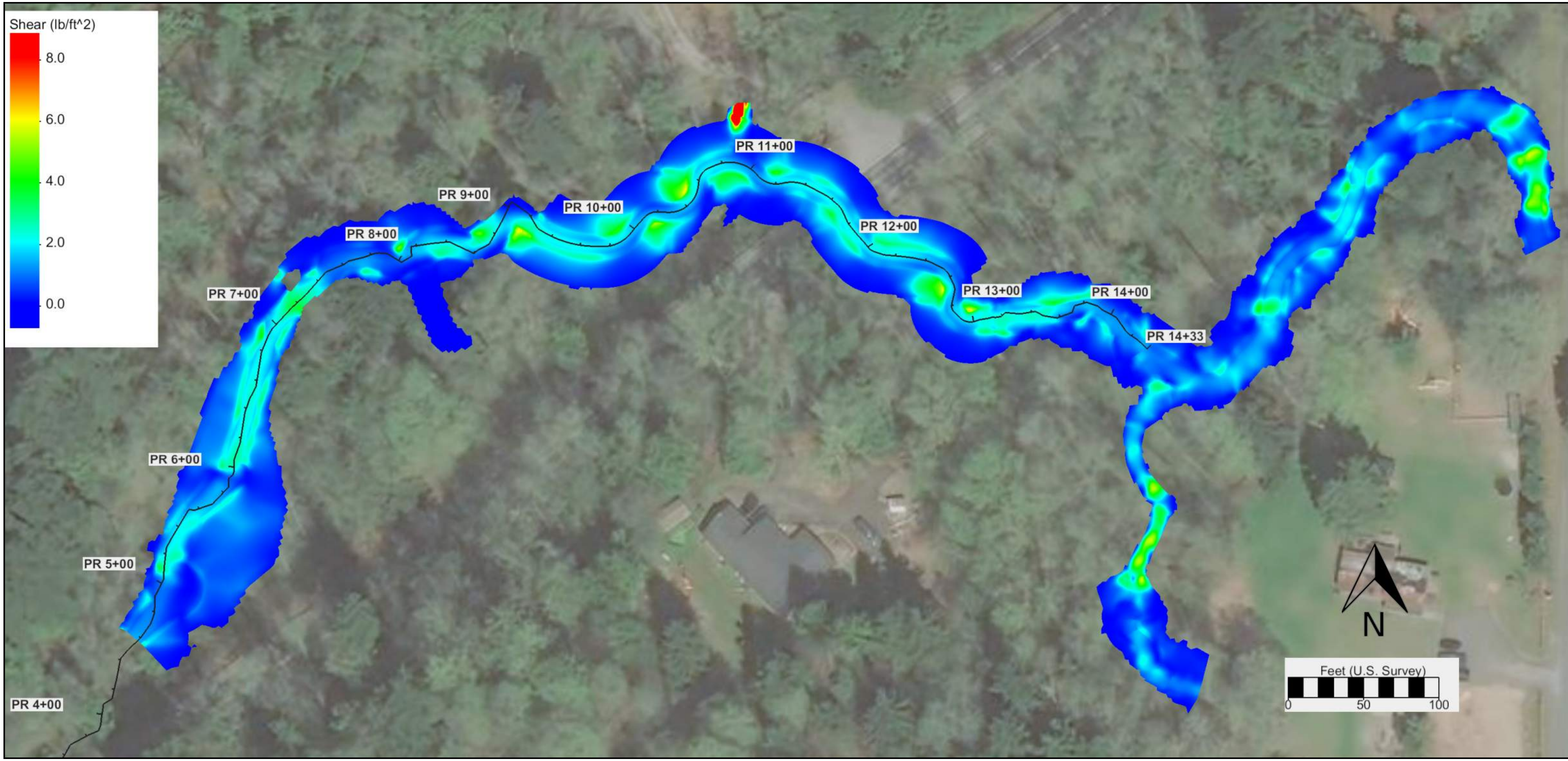
NATURAL CONDITION 500-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



SHEAR

NATURAL CONDITION 500-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

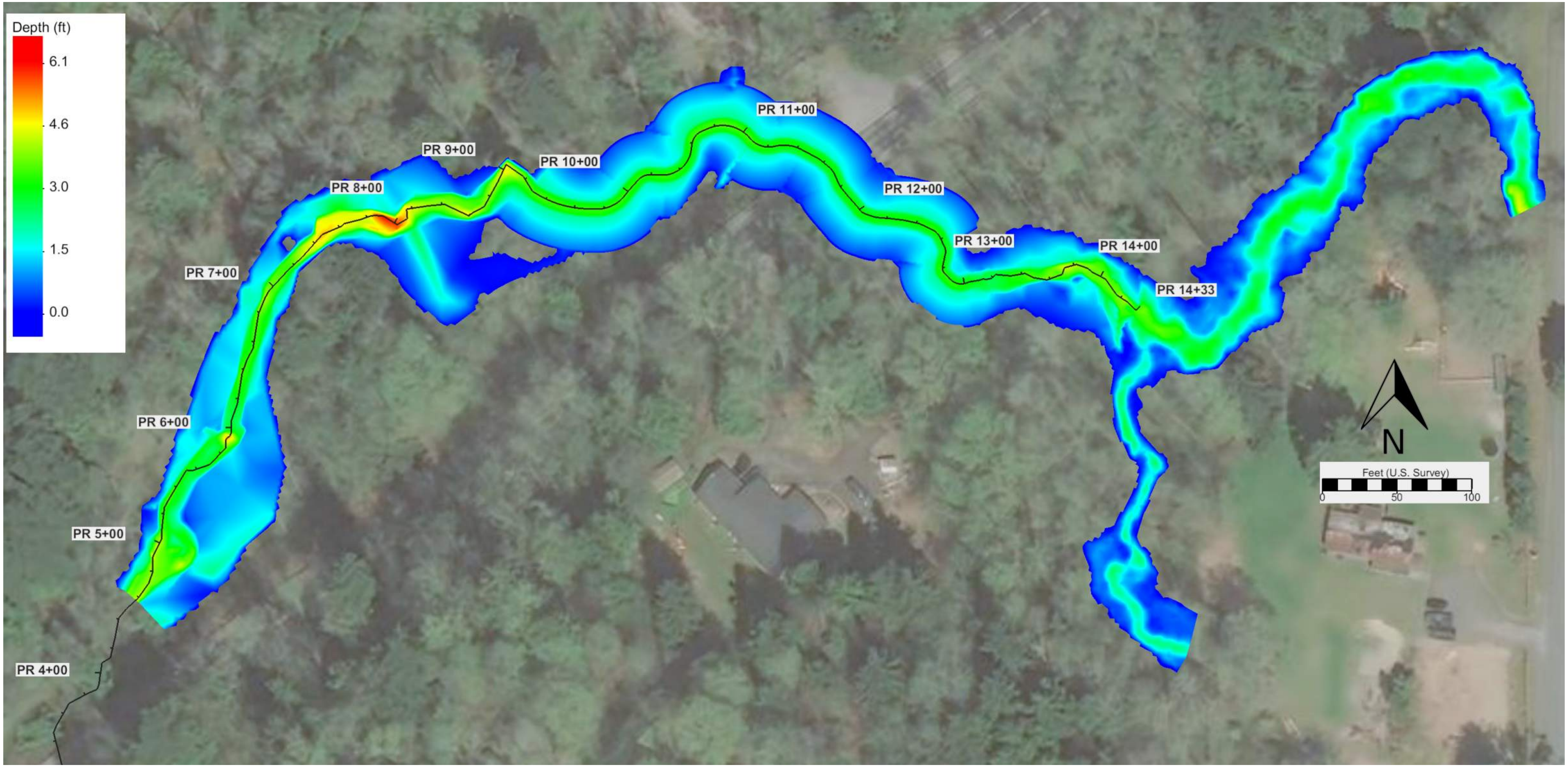
SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN

WATER SURFACE ELEVATION

NATURAL CONDITION 2080 100-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK



DEPTH

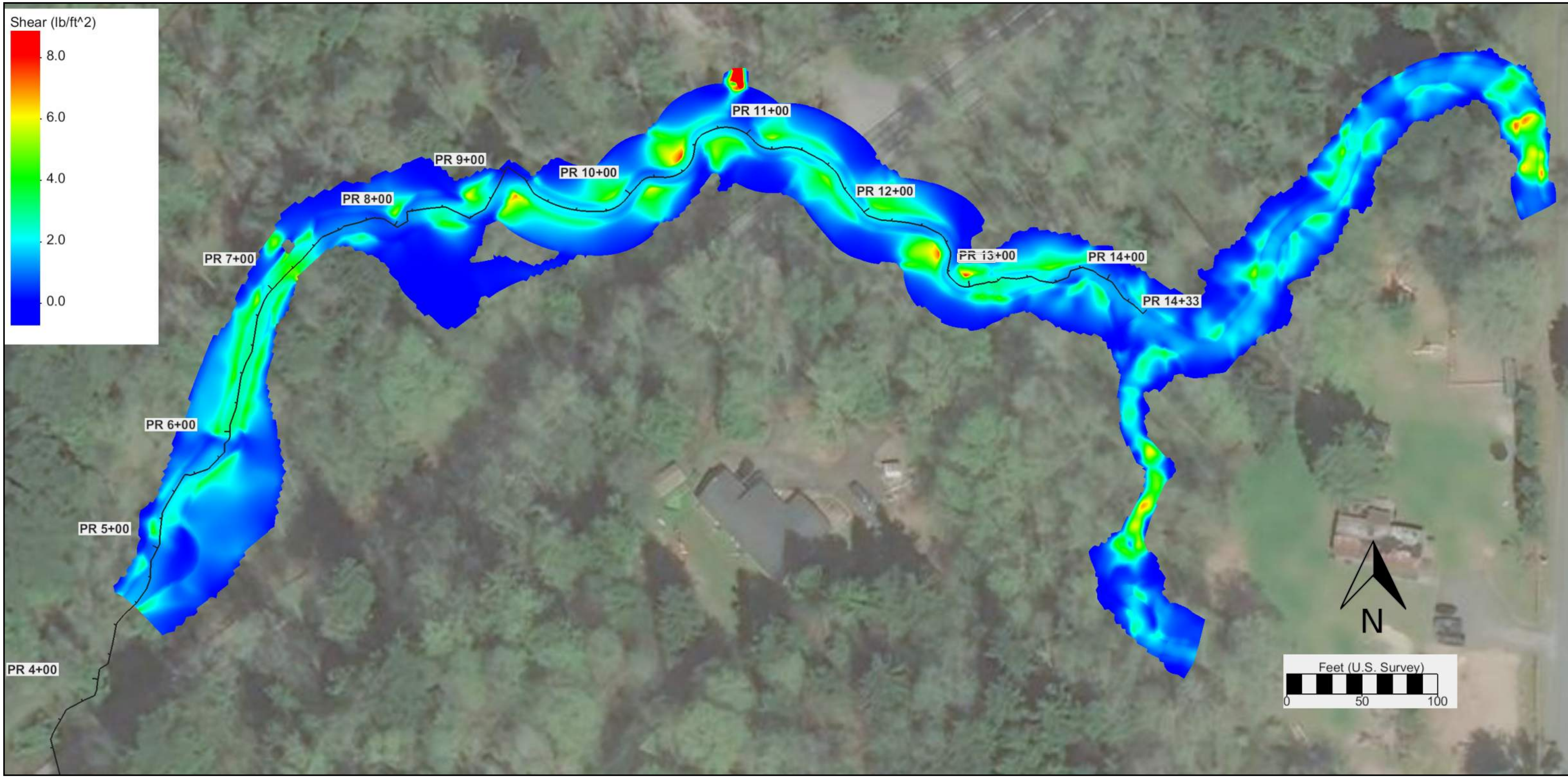
NATURAL CONDITION 2080 100-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

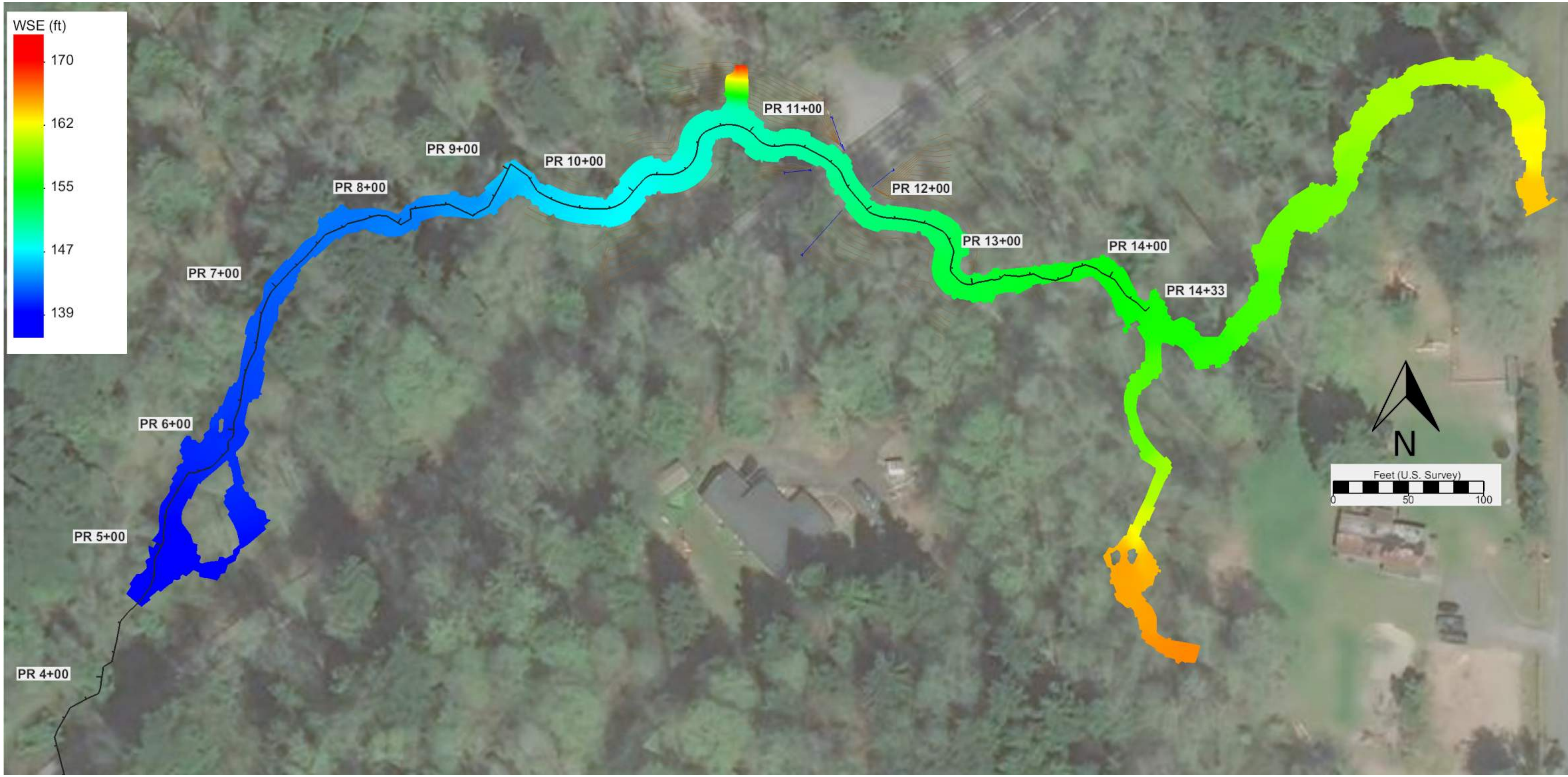
SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



SHEAR

NATURAL CONDITION 2080 100-YEAR EVENT



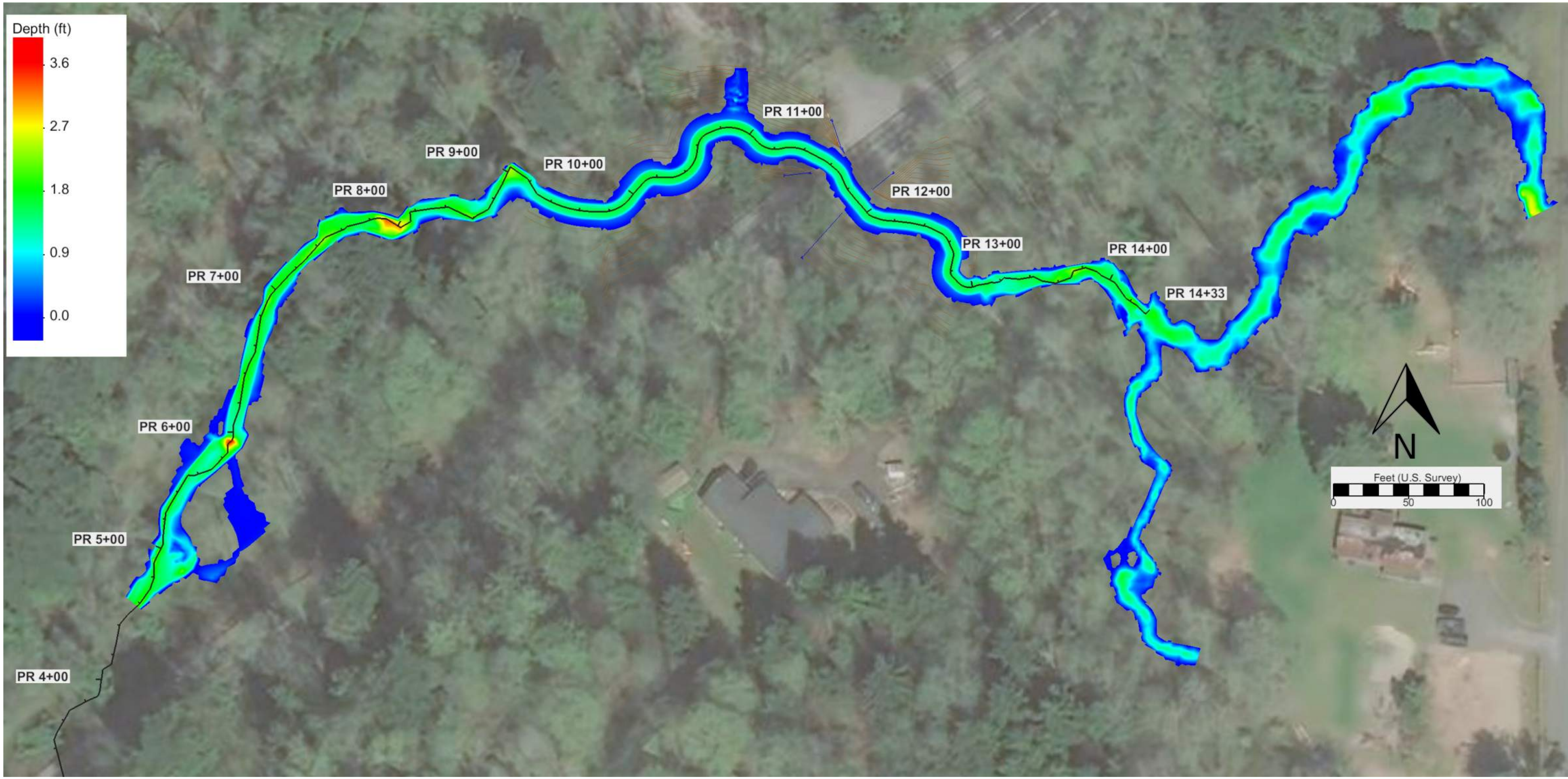
WATER SURFACE ELEVATION

PROPOSED CONDITION 2-YEAR EVENT

PACE An Engineering Services Company

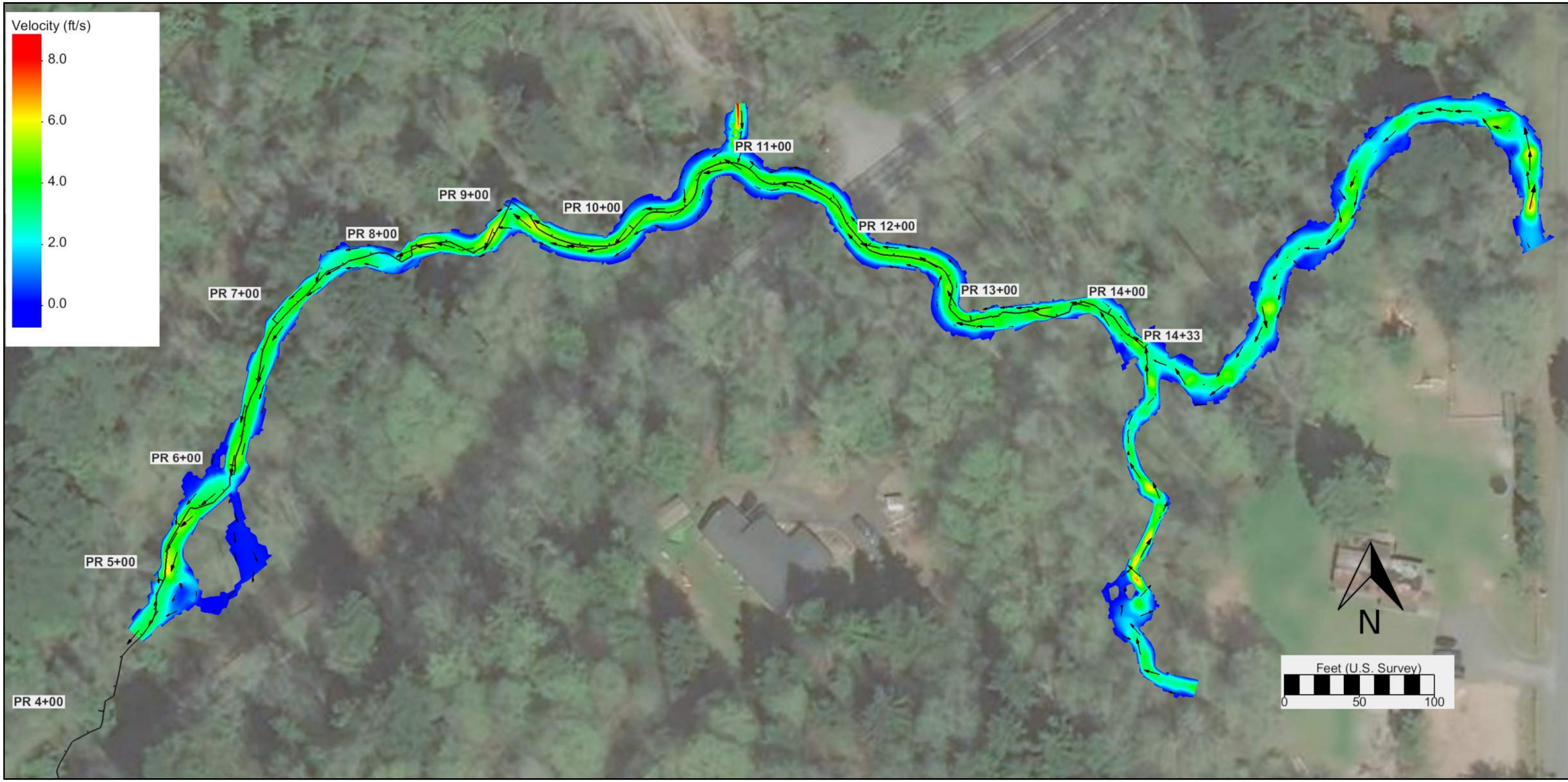
Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK



DEPTH

PROPOSED CONDITION 2-YEAR EVENT



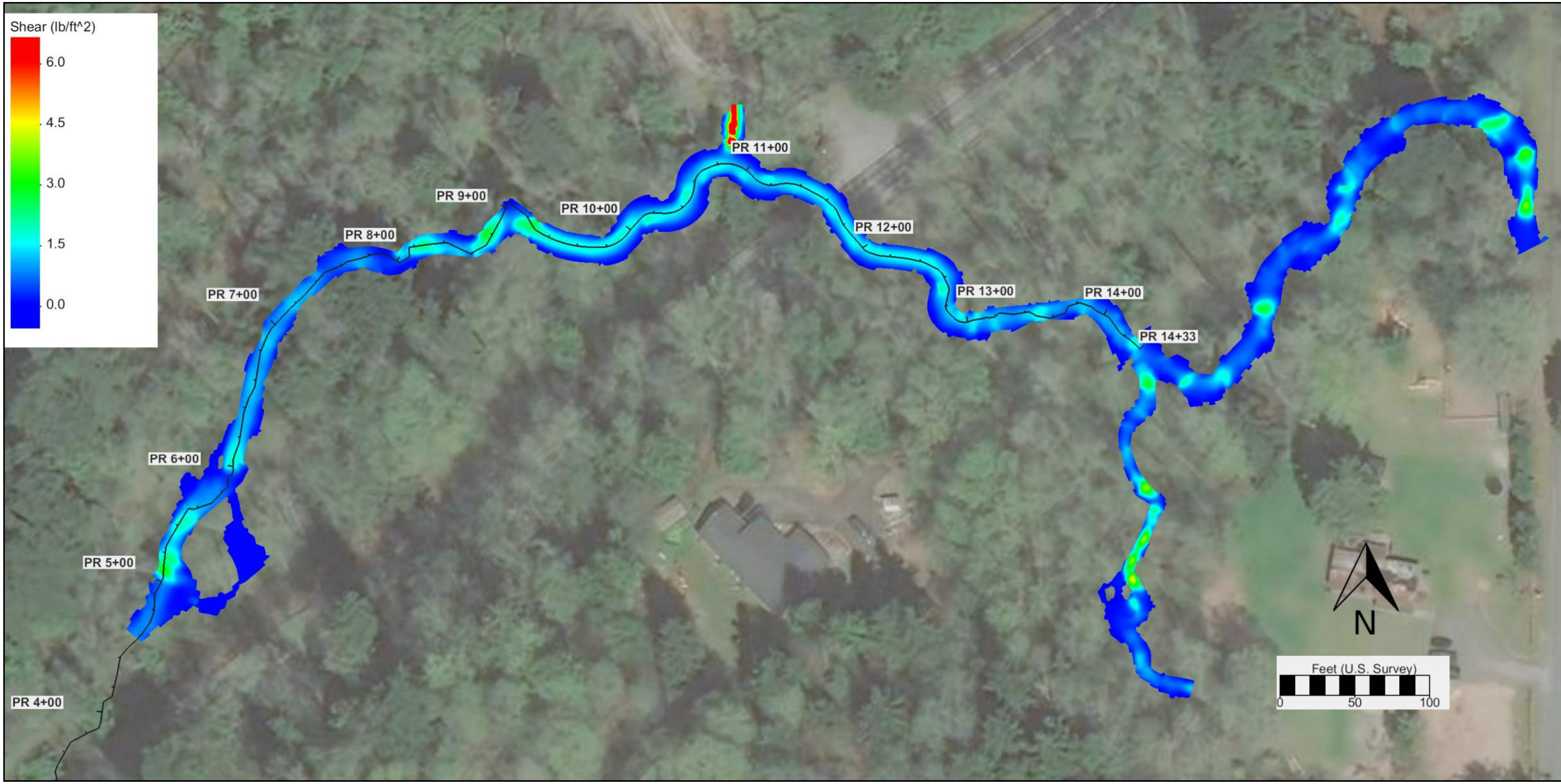
VELOCITY

PROPOSED CONDITION 2-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK



SHEAR

PACE An Engineering Services Company

Washington State Department of Transportation

PROPOSED CONDITION 2-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

WATER SURFACE ELEVATION

PROPOSED CONDITION 100-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN

DEPTH

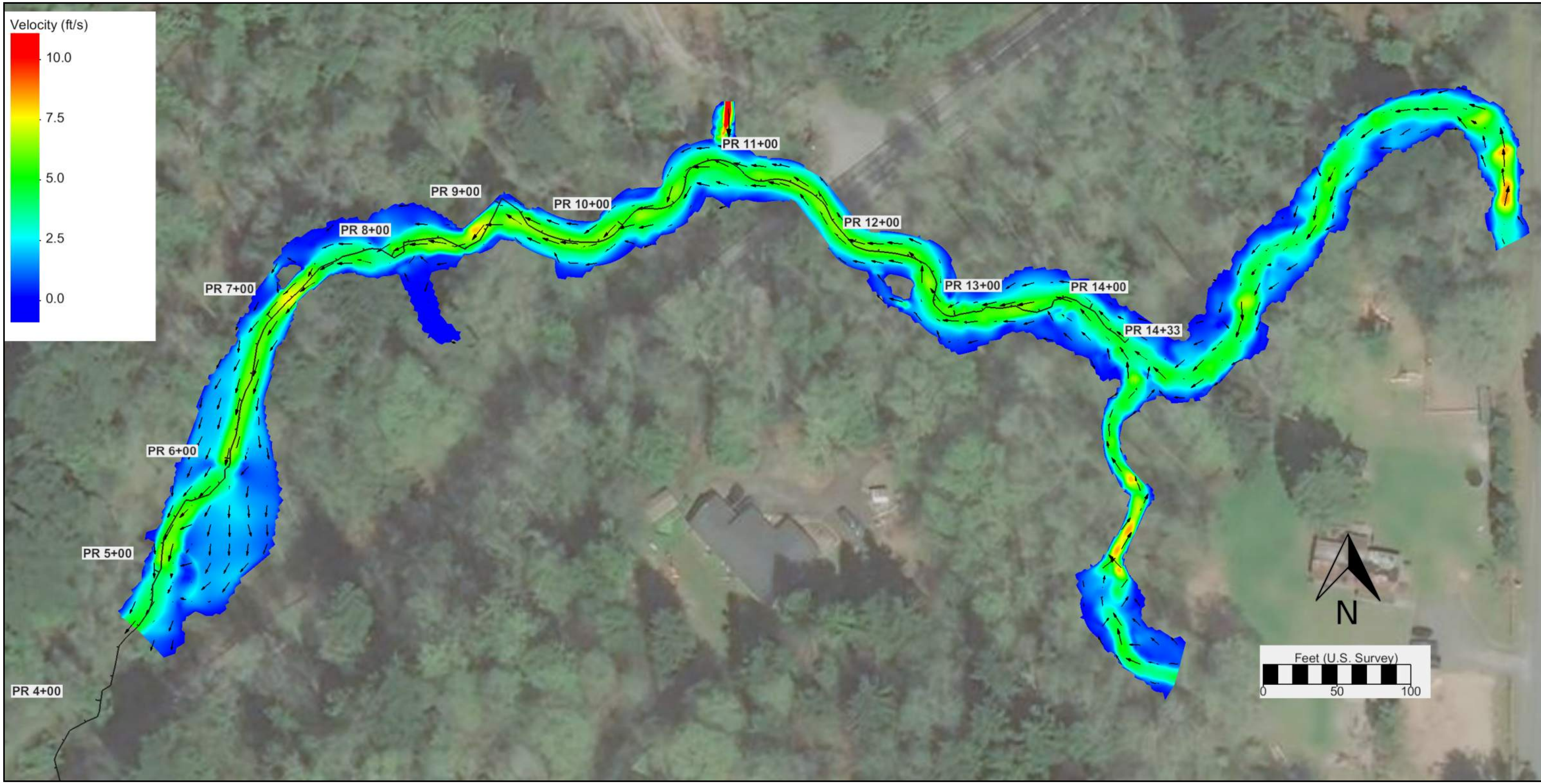
PROPOSED CONDITION 100-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



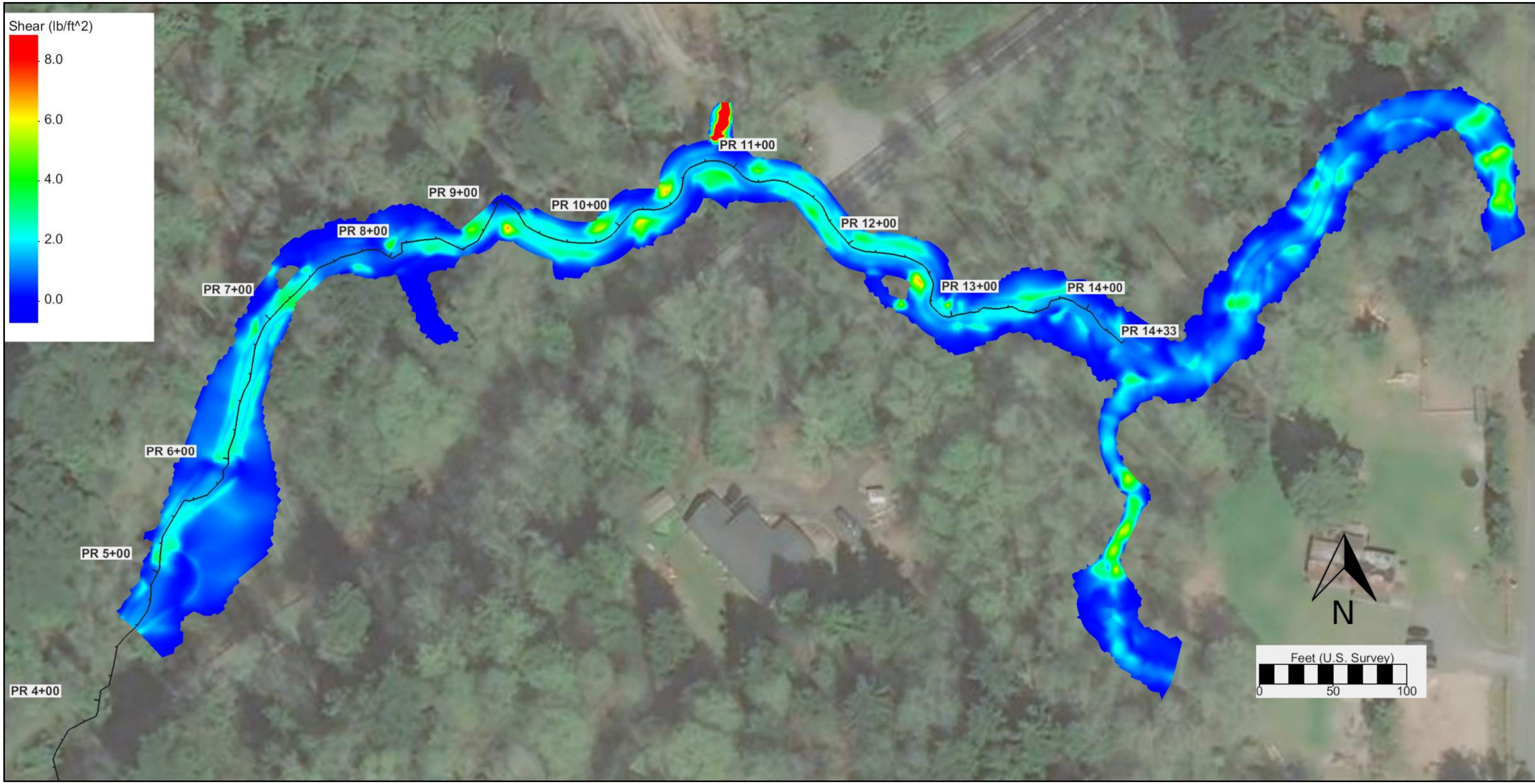
VELOCITY

PROPOSED CONDITION 100-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK



PACE An Engineering Services Company

Washington State Department of Transportation

SHEAR

PROPOSED CONDITION 100-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN

WATER SURFACE ELEVATION

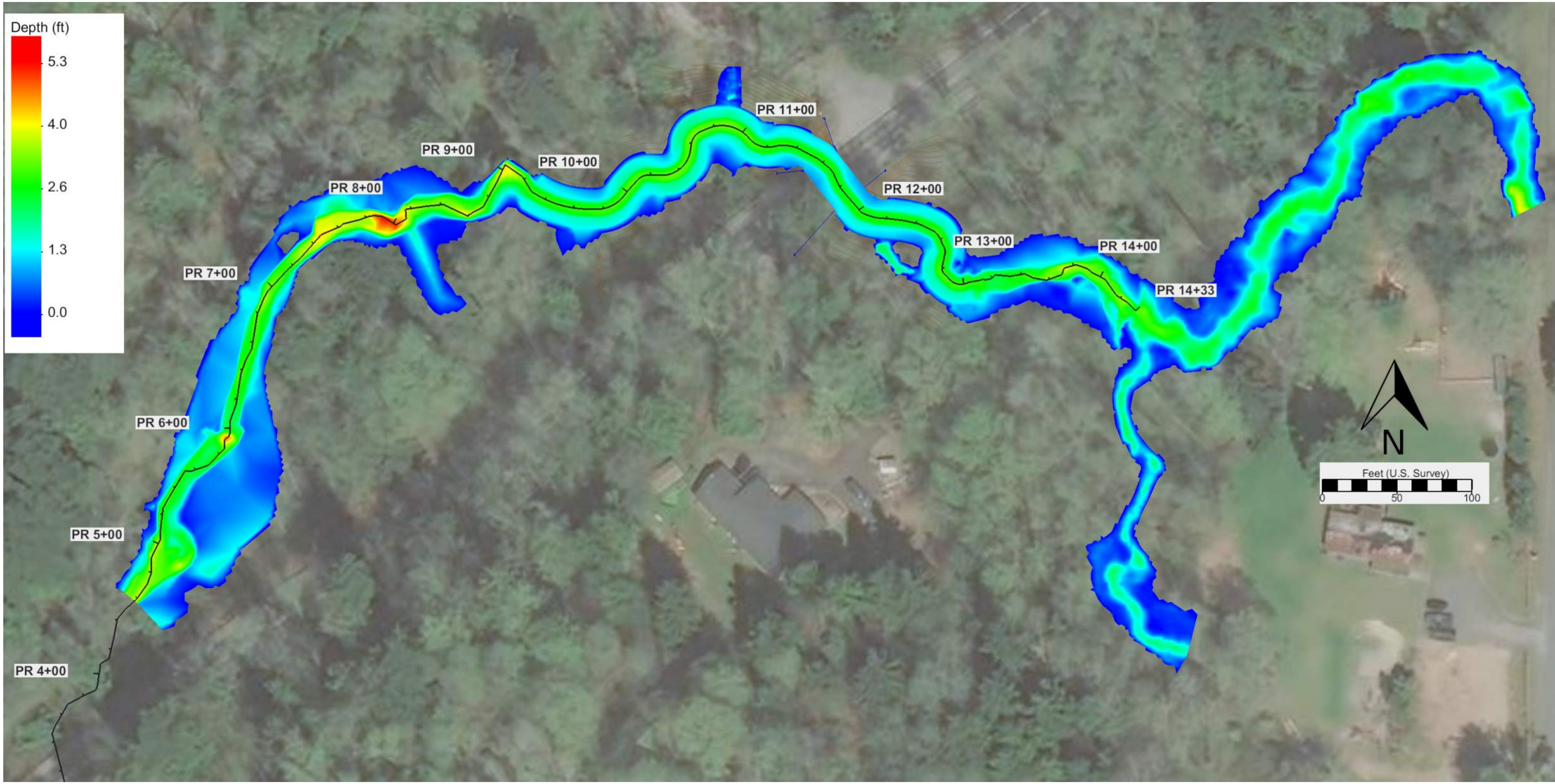
PROPOSED CONDITION 500-YEAR EVENT

PACE
An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



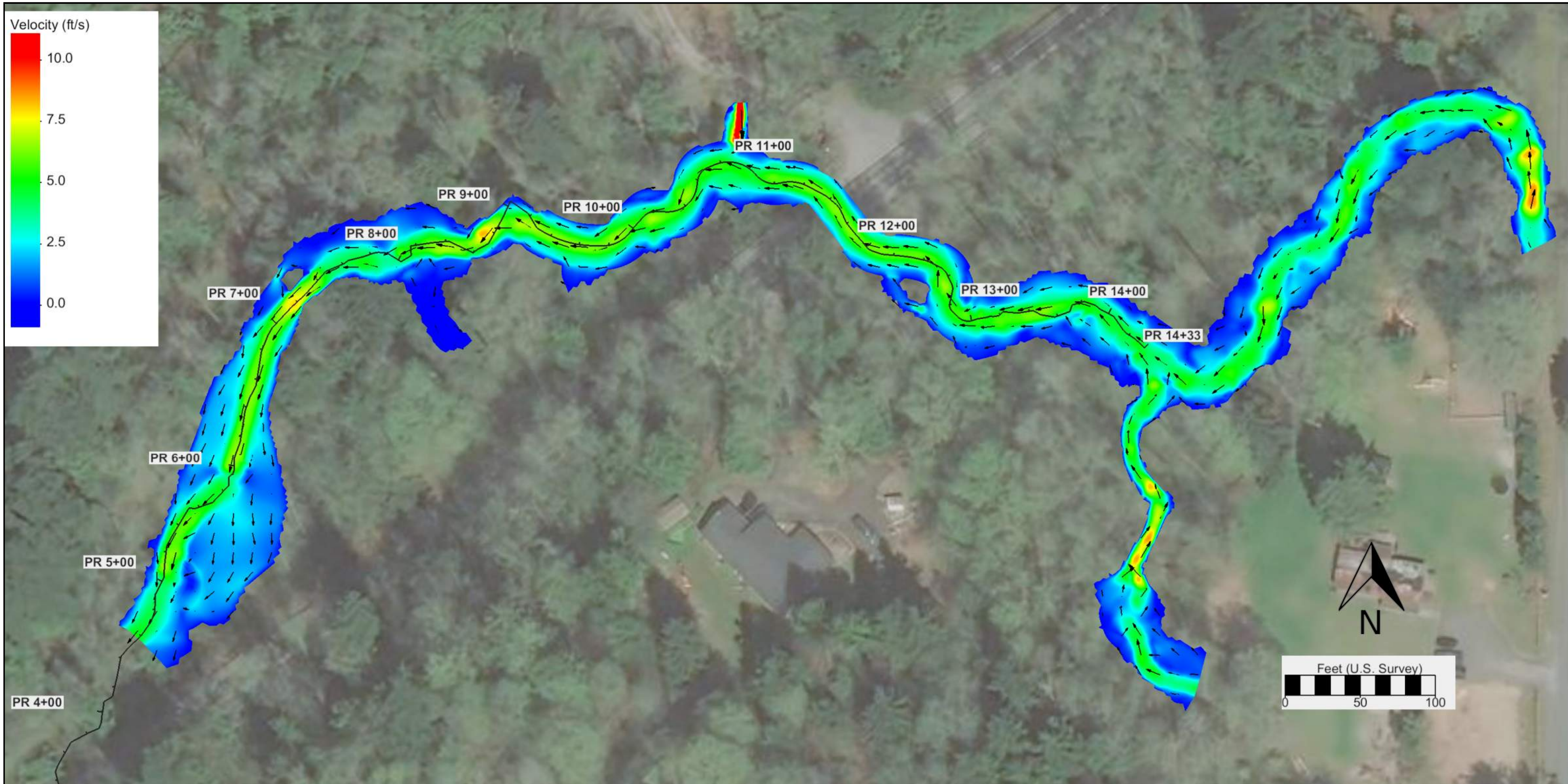
DEPTH

PACE An Engineering Services Company

Washington State Department of Transportation

PROPOSED CONDITION 500-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK



VELOCITY

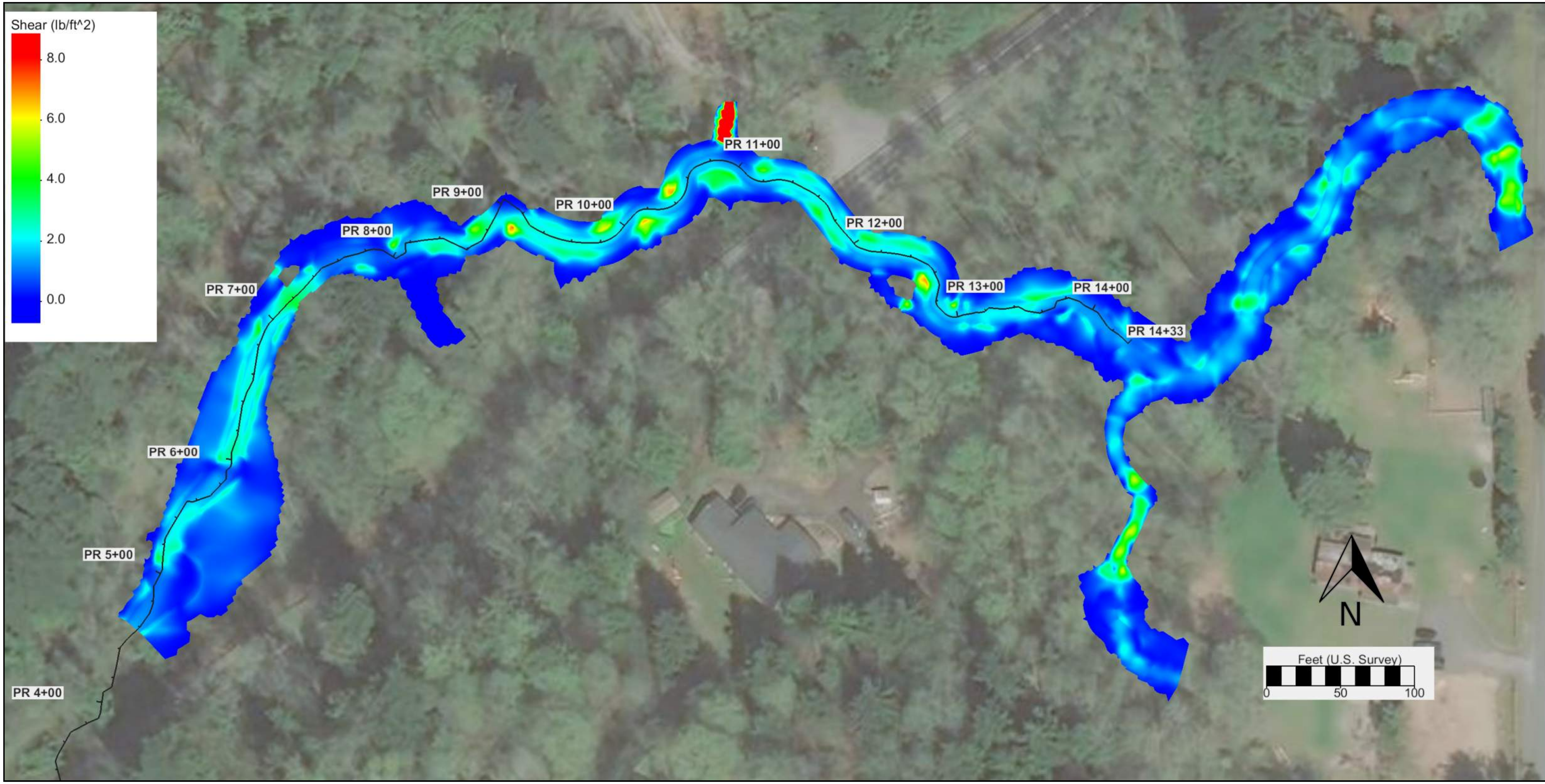
PROPOSED CONDITION 500-YEAR EVENT

PACE
An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



SHEAR

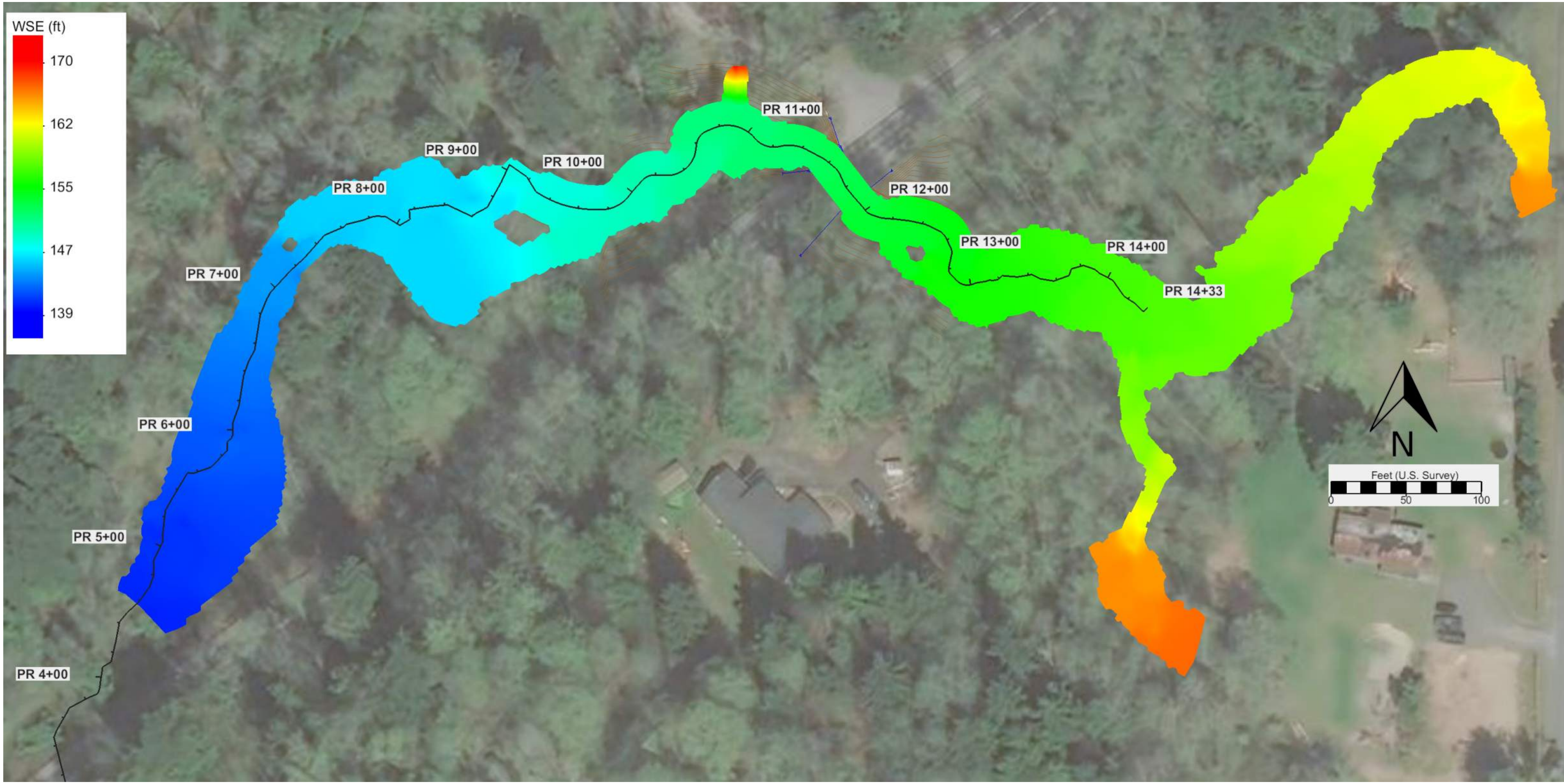
PACE An Engineering Services Company

Washington State Department of Transportation

PROPOSED CONDITION 500-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



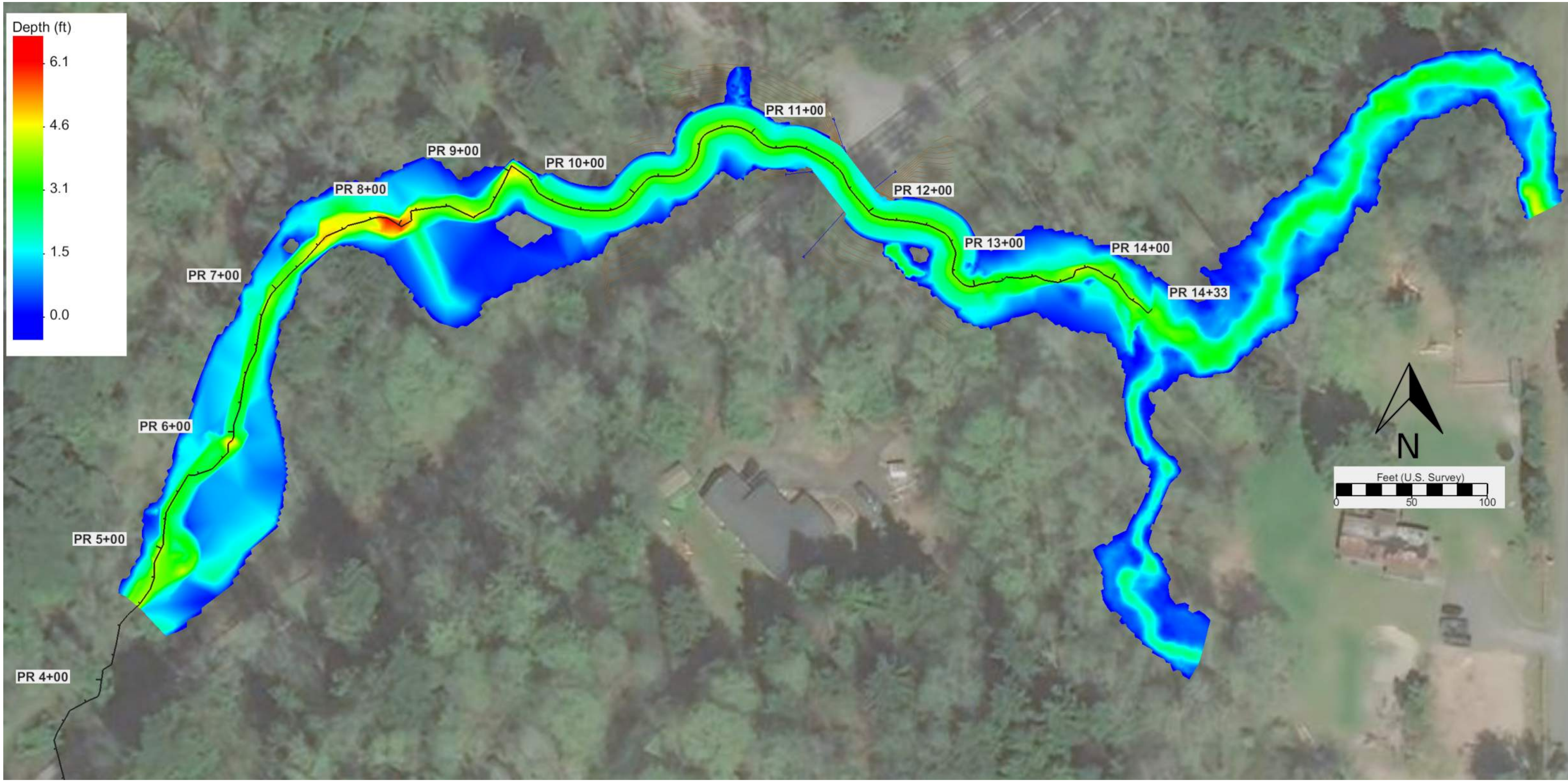
WATER SURFACE ELEVATION

PROPOSED CONDITION 2080 100-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK



DEPTH

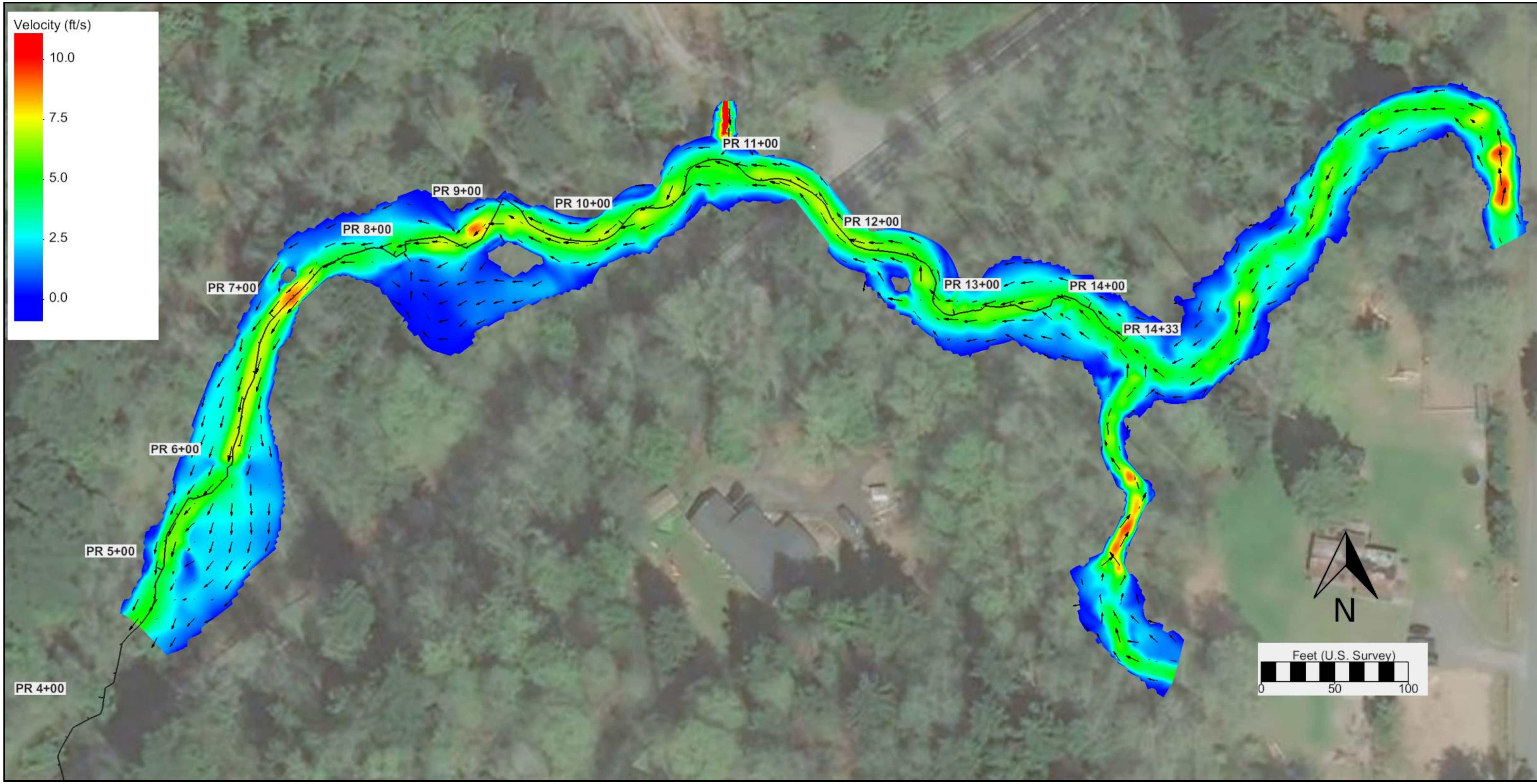
PROPOSED CONDITION 2080 100-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



VELOCITY

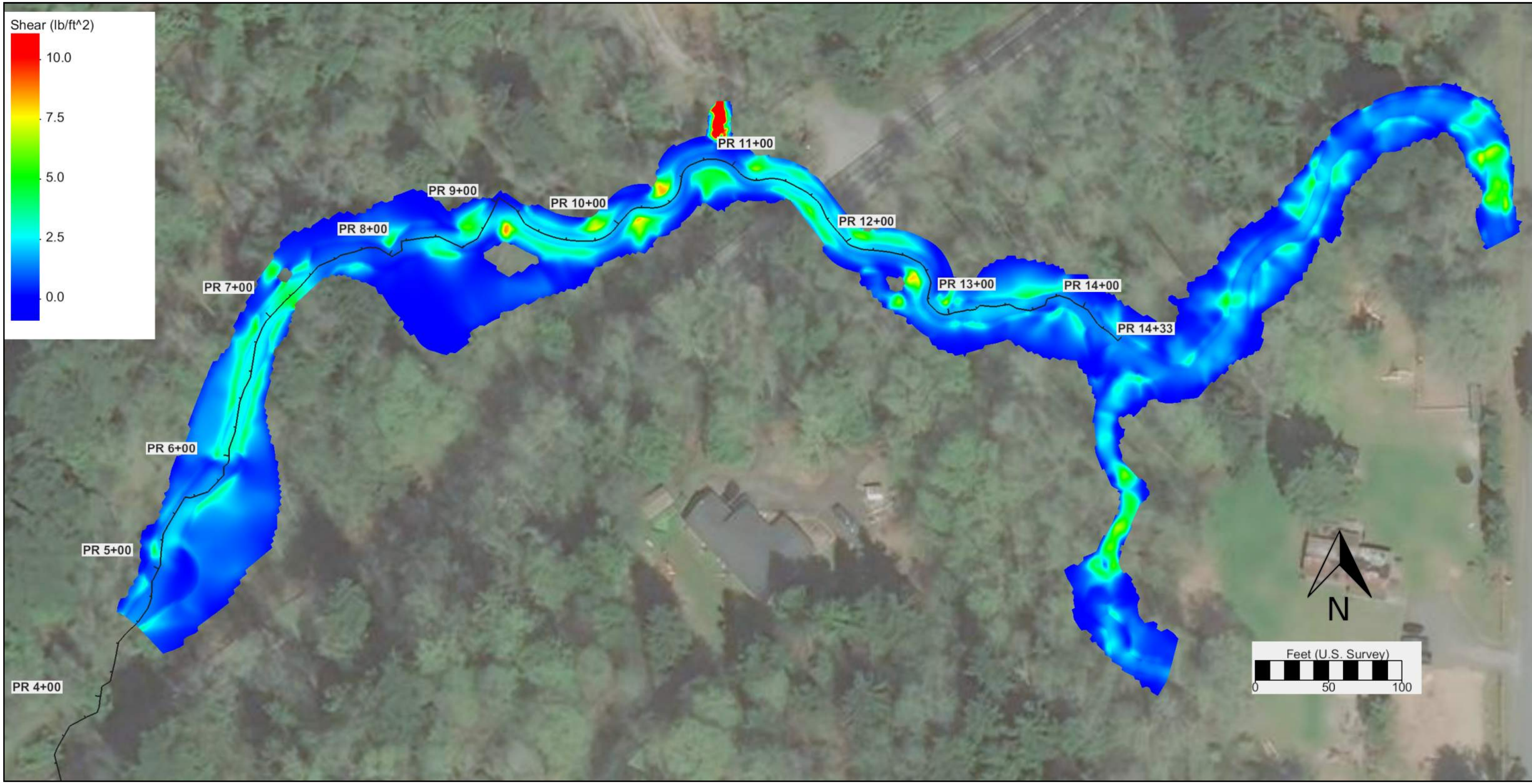
PACE An Engineering Services Company

Washington State Department of Transportation

PROPOSED CONDITION 2080 100-YEAR EVENT

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN



SHEAR

PROPOSED CONDITION 2080 100-YEAR EVENT

PACE An Engineering Services Company

Washington State Department of Transportation

SR 307 MP 1.45 NORTHEAST DOGFISH CREEK TO DOGFISH CREEK

PRELIMINARY HYDRAULIC DESIGN

Appendix I: SRH-2D Model Stability and Continuity

Figure 1. Existing Condition Monitor Location

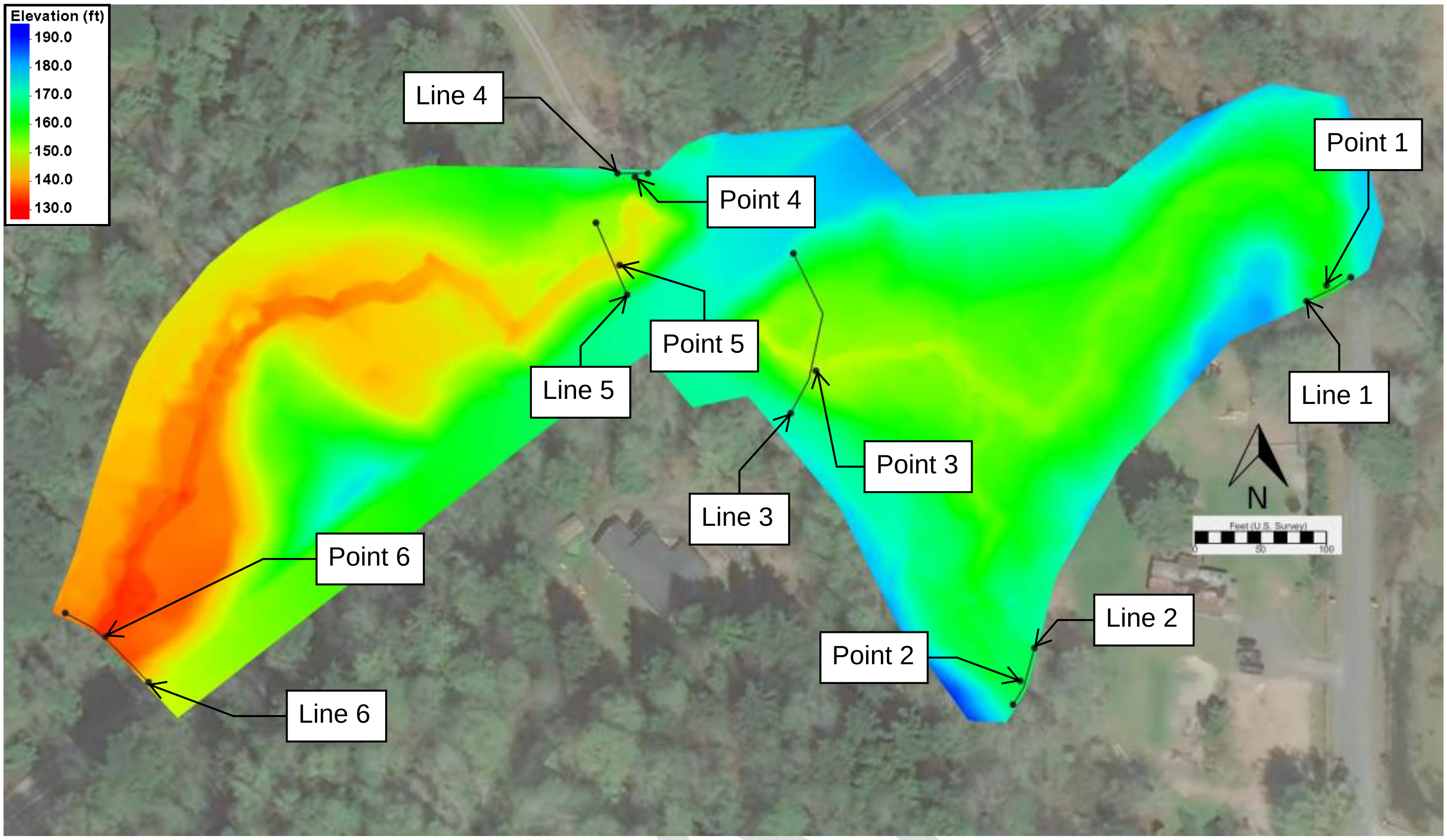


Figure 2. Natural Condition Monitor Location

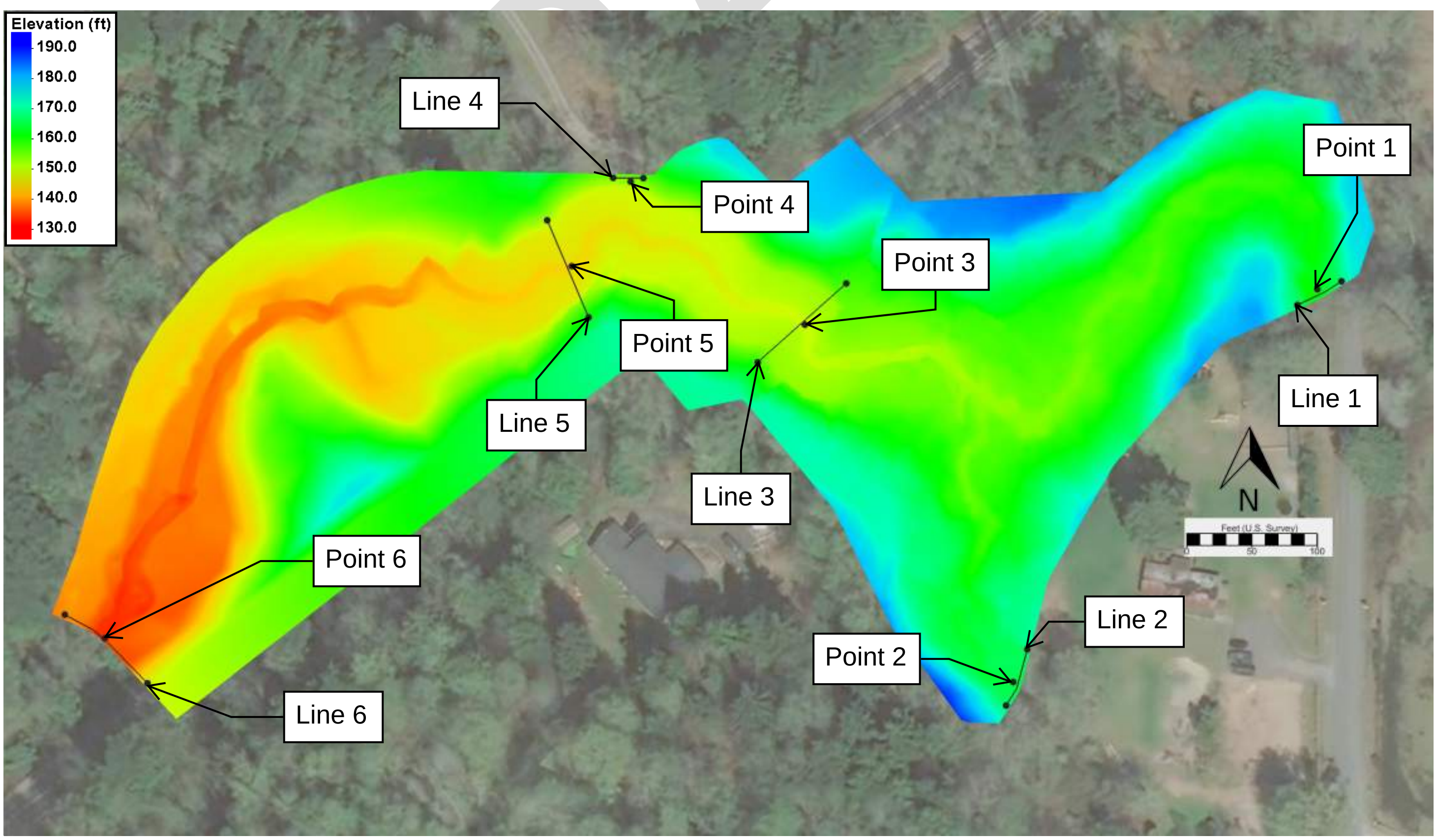


Figure 3. Proposed Condition Monitor Location

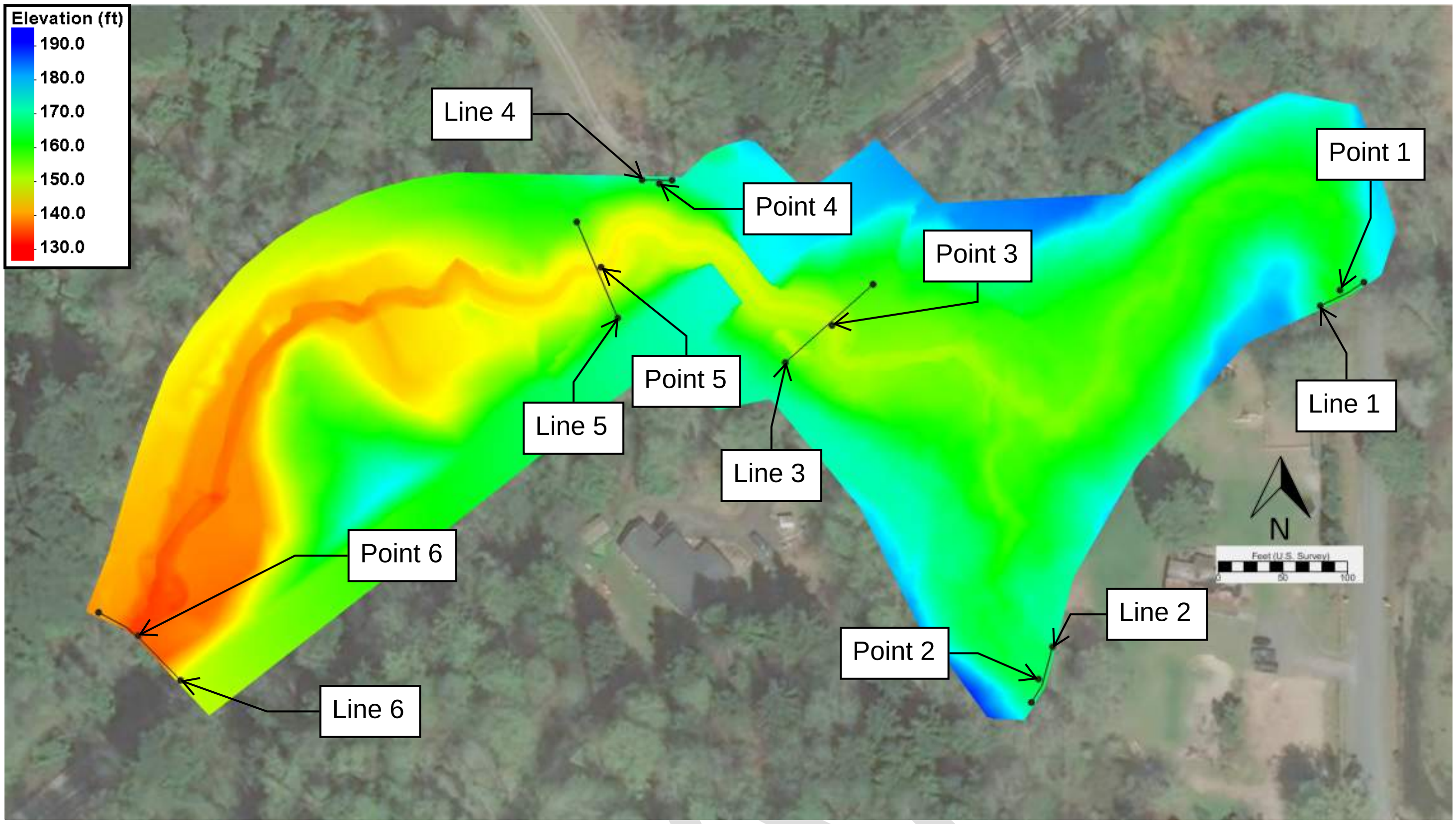


Figure 4. Existing 2-year Condition Monitor Point Plot

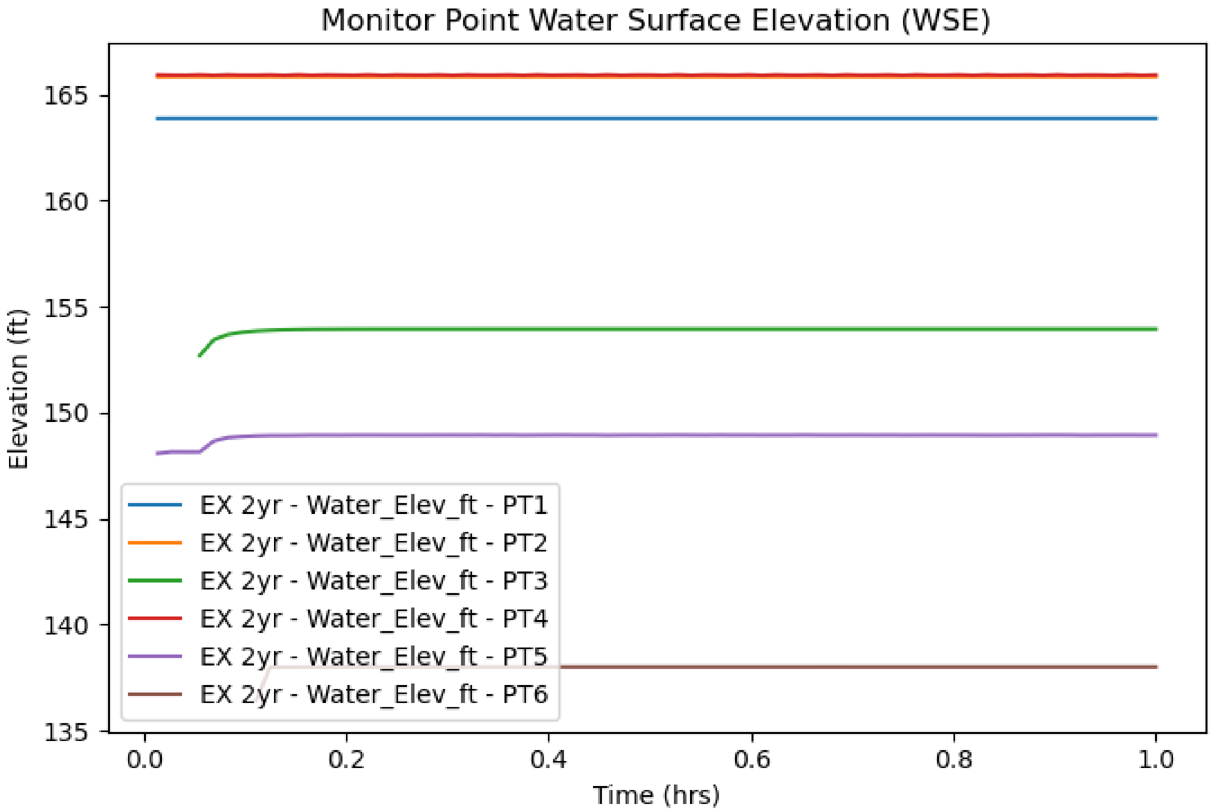


Figure 5. Existing 2-year Condition Monitor Line Plot

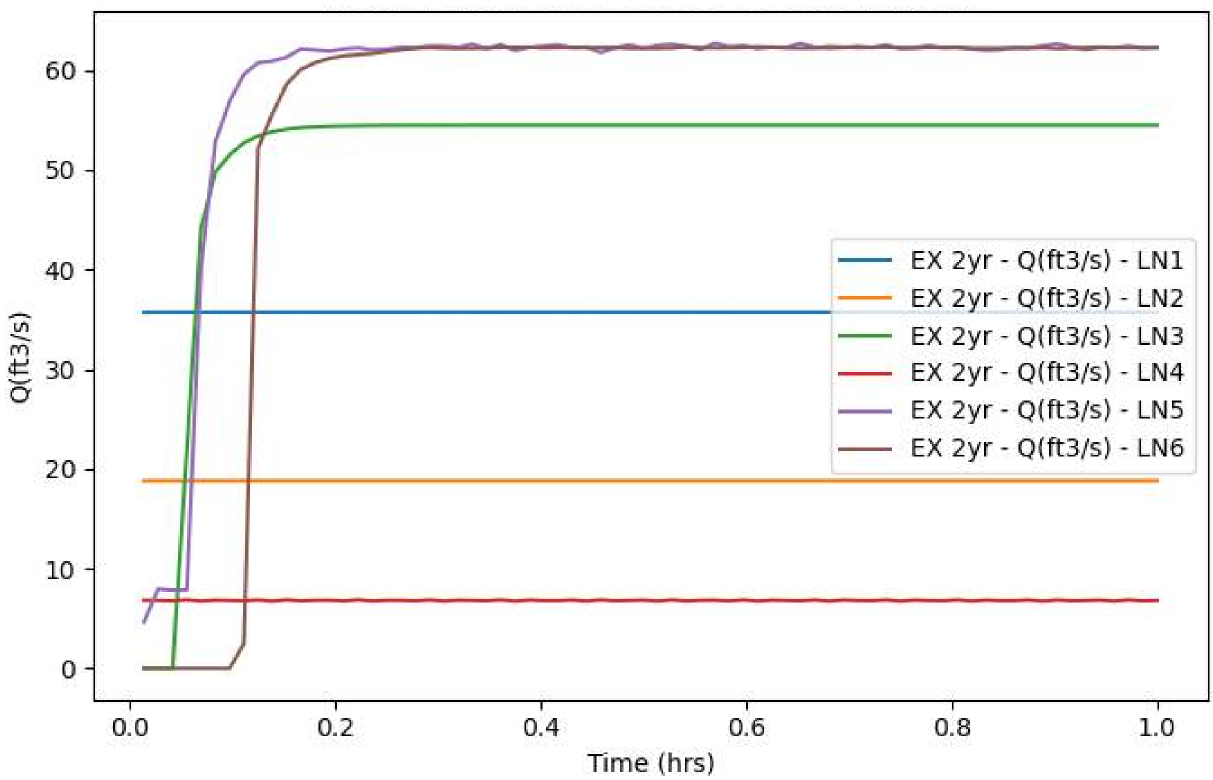


Figure 6. Existing 100-year Condition Monitor Point Plot

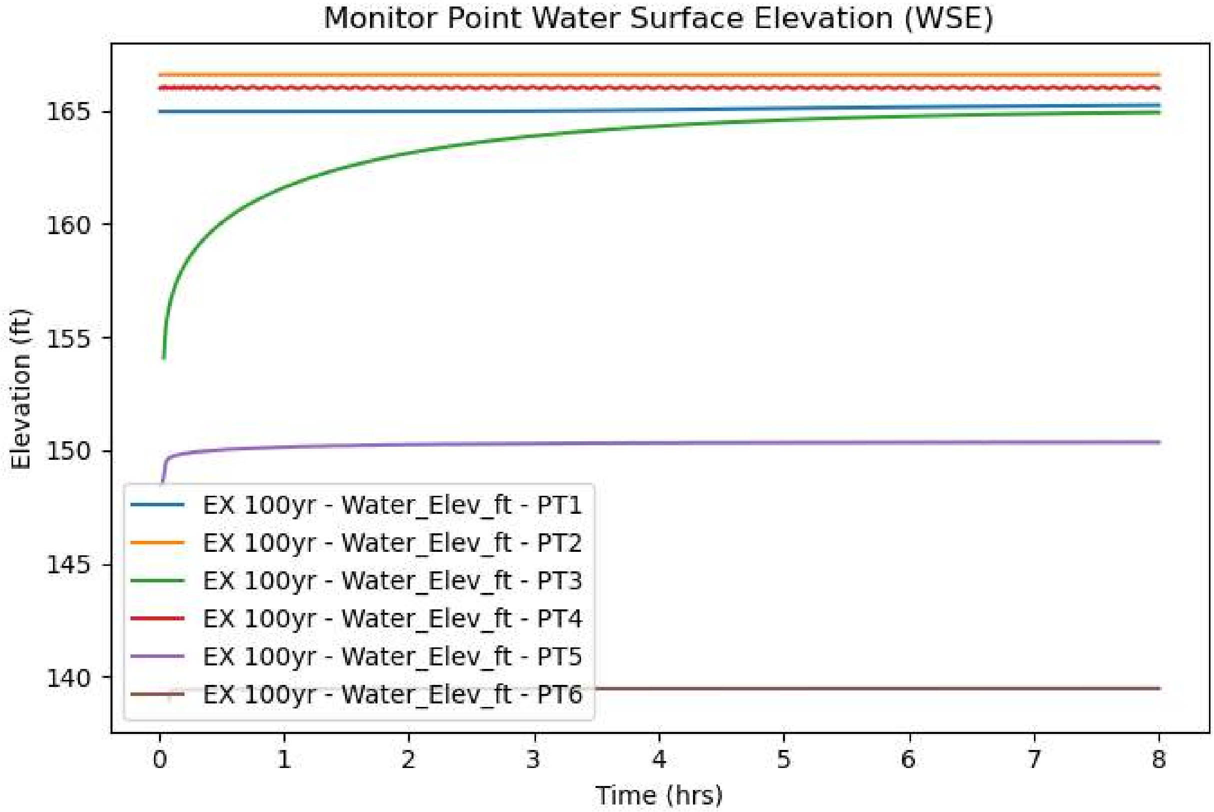


Figure 7. Existing 100-year Condition Monitor Line Plot

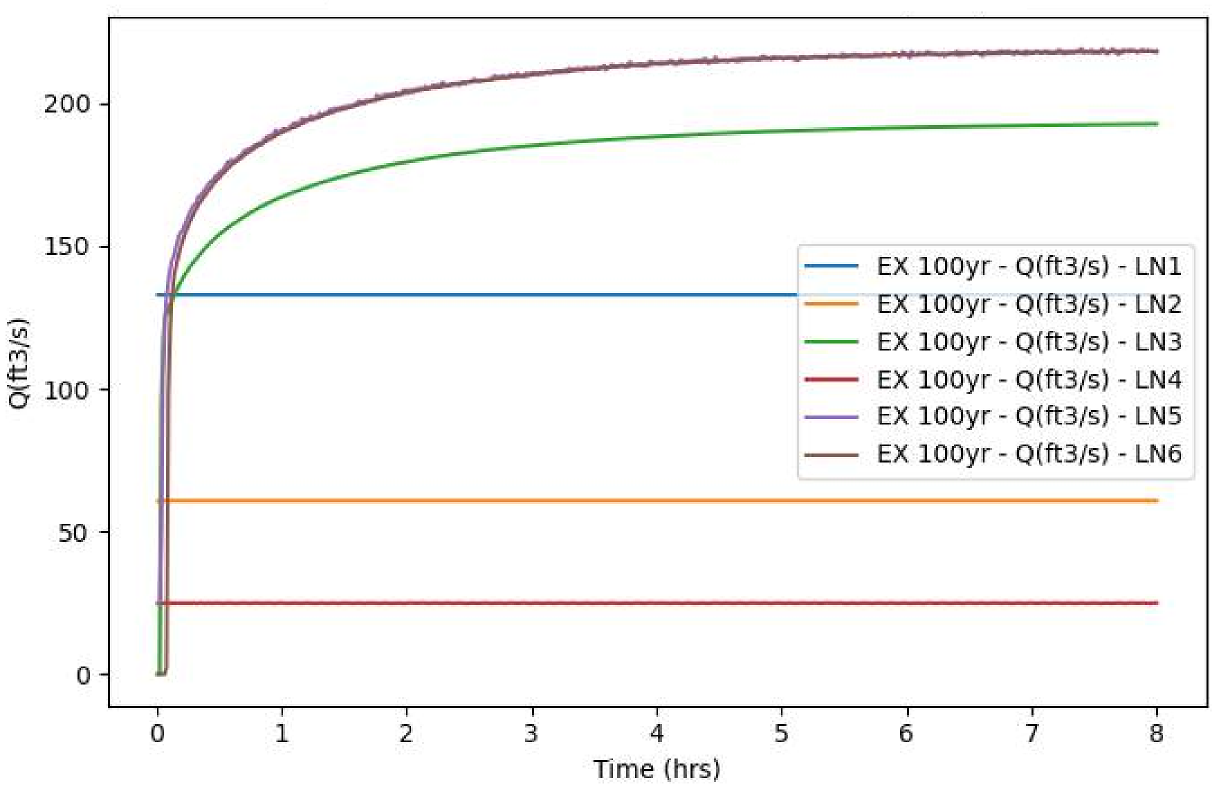


Figure 8. Existing 500-year Condition Monitor Point Plot

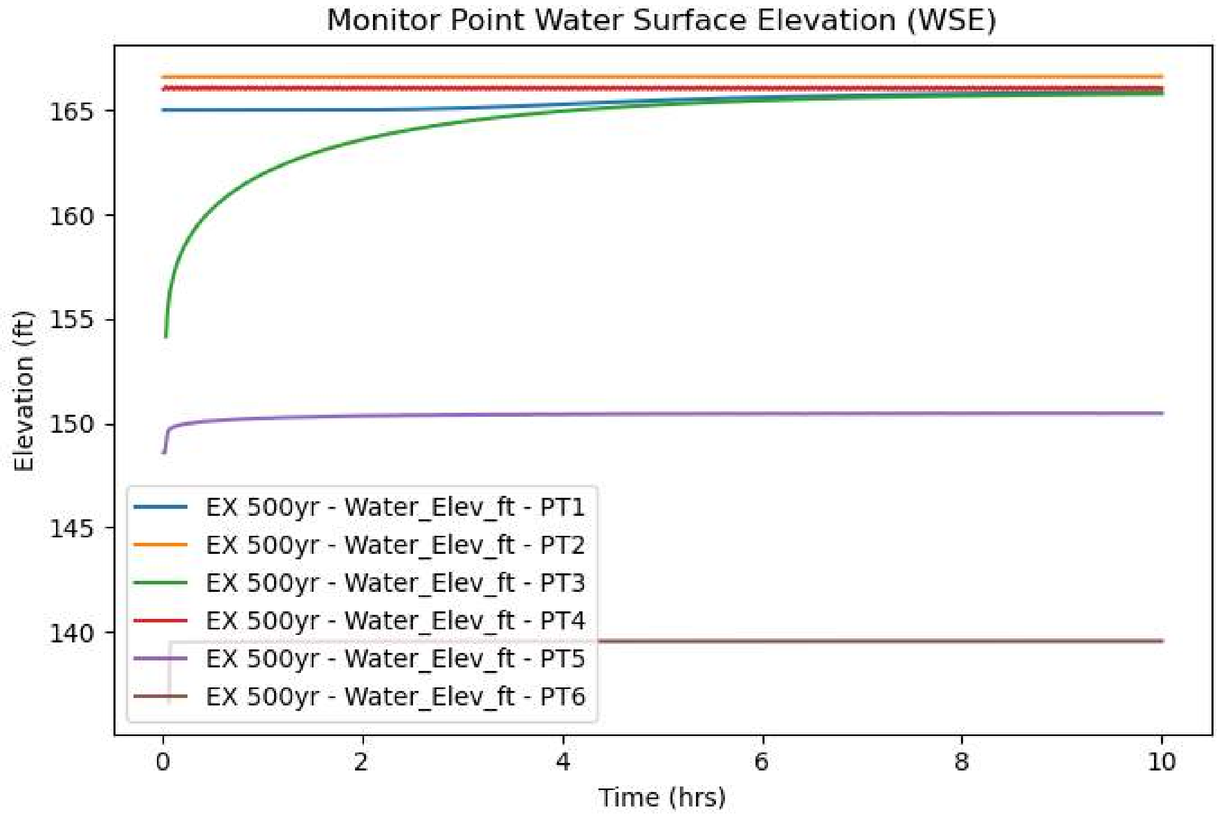


Figure 9. Existing 500-year Condition Monitor Line Plot

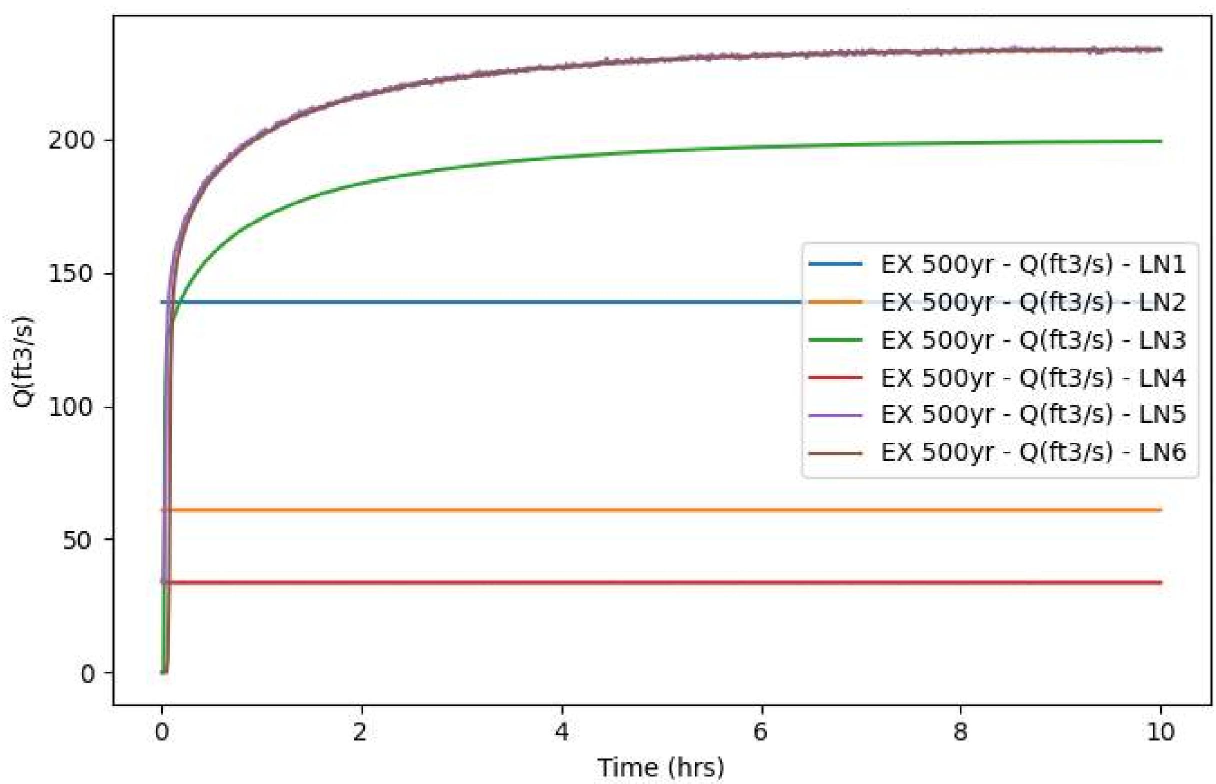


Figure 10. Natural 2-year Condition Monitor Point Plot

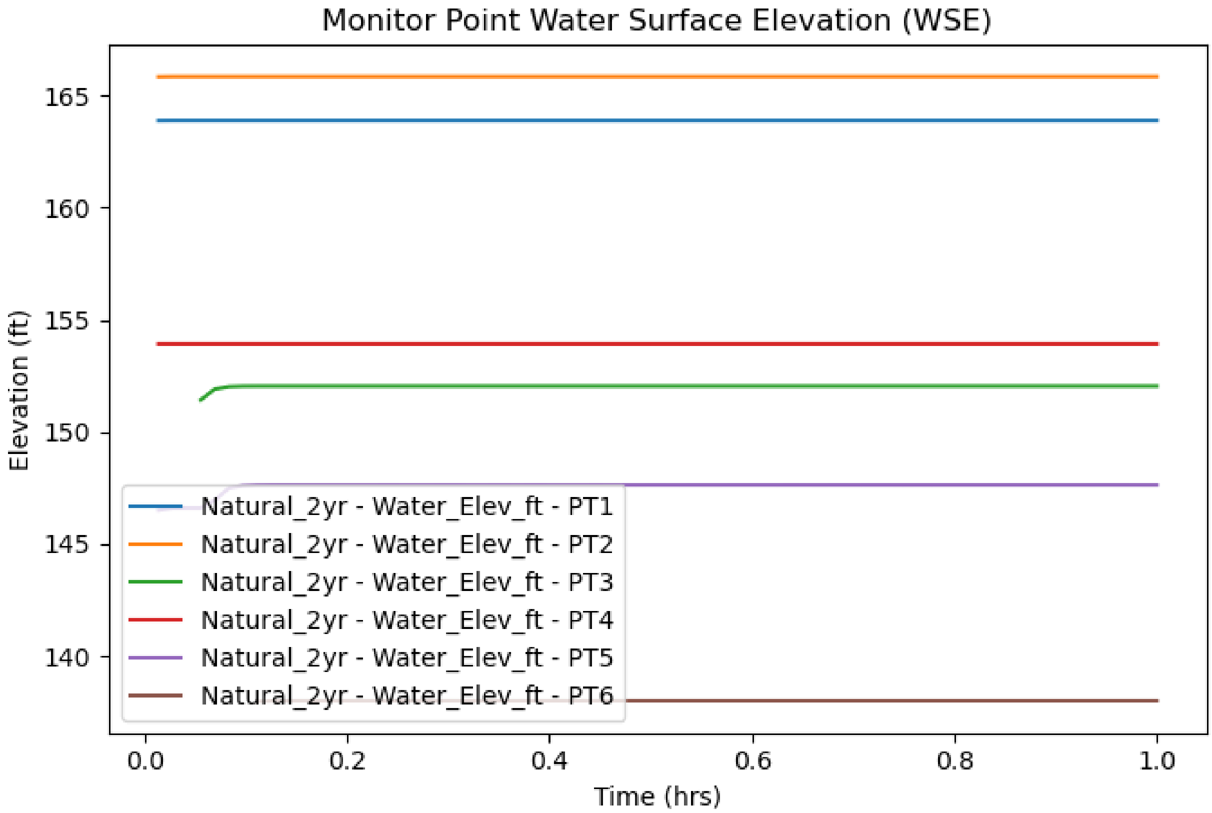


Figure 11. Natural 2-year Condition Monitor Line Plot

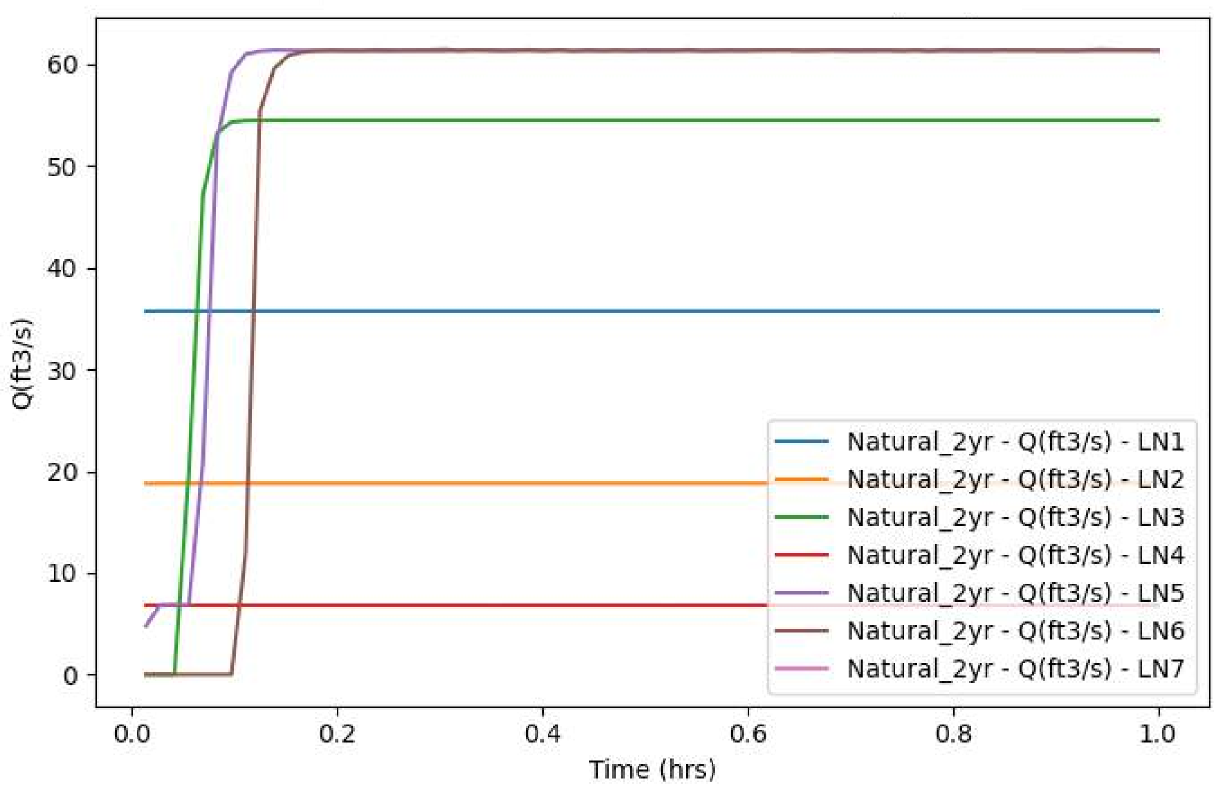


Figure 12. Natural 100-year Condition Monitor Point Plot

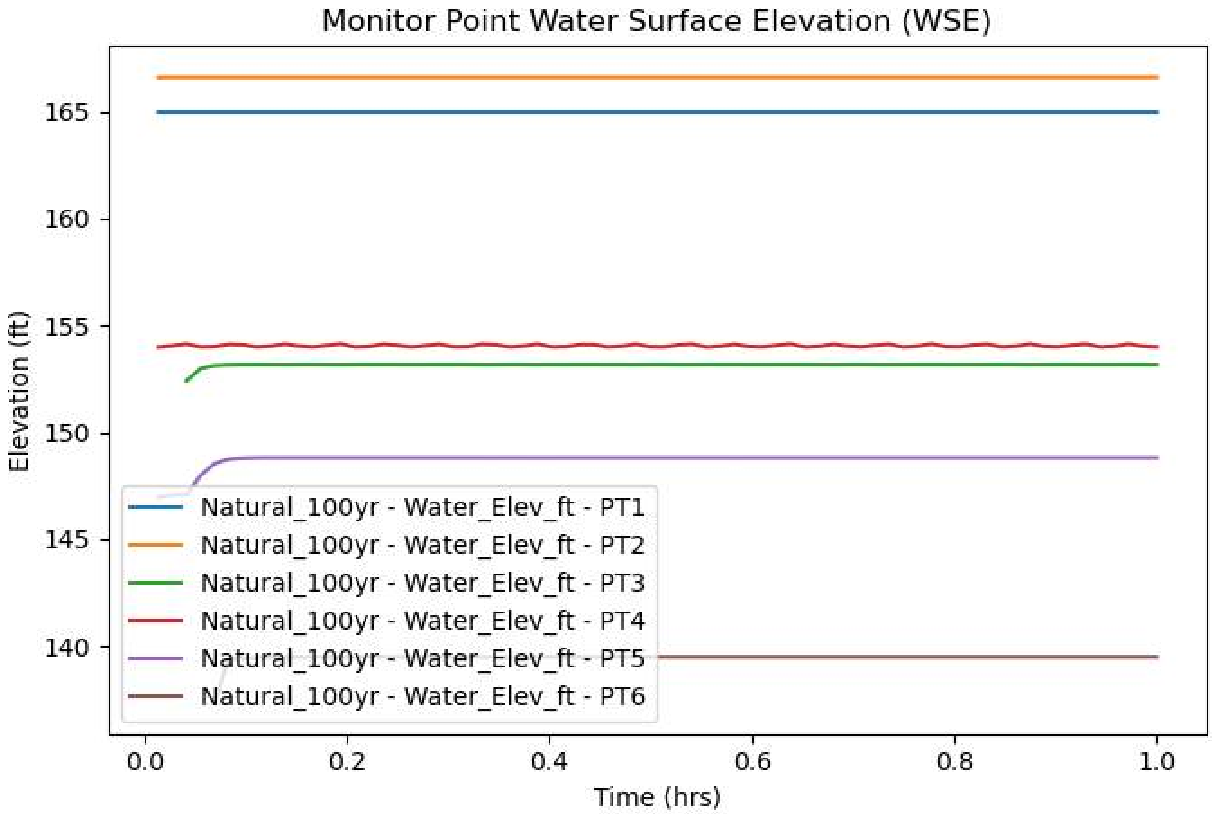


Figure 13. Natural 100-year Condition Monitor Line Plot

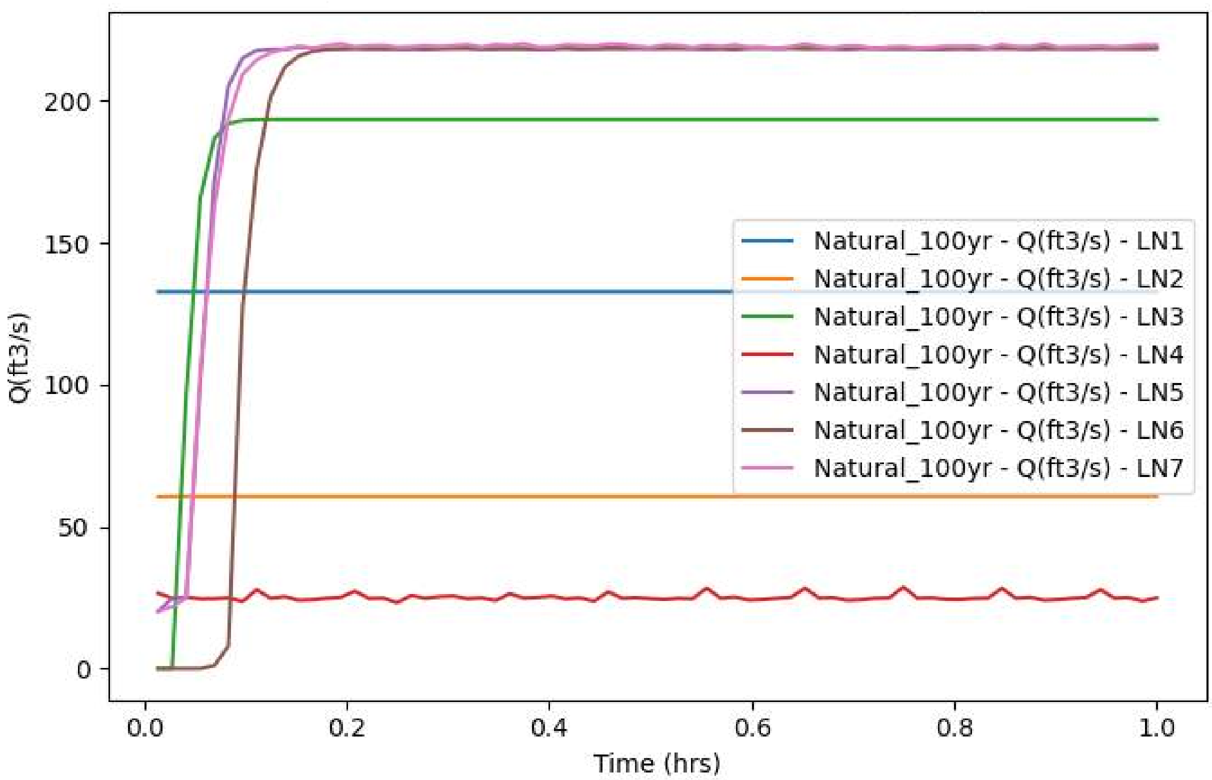


Figure 14. Natural 500-year Condition Monitor Point Plot

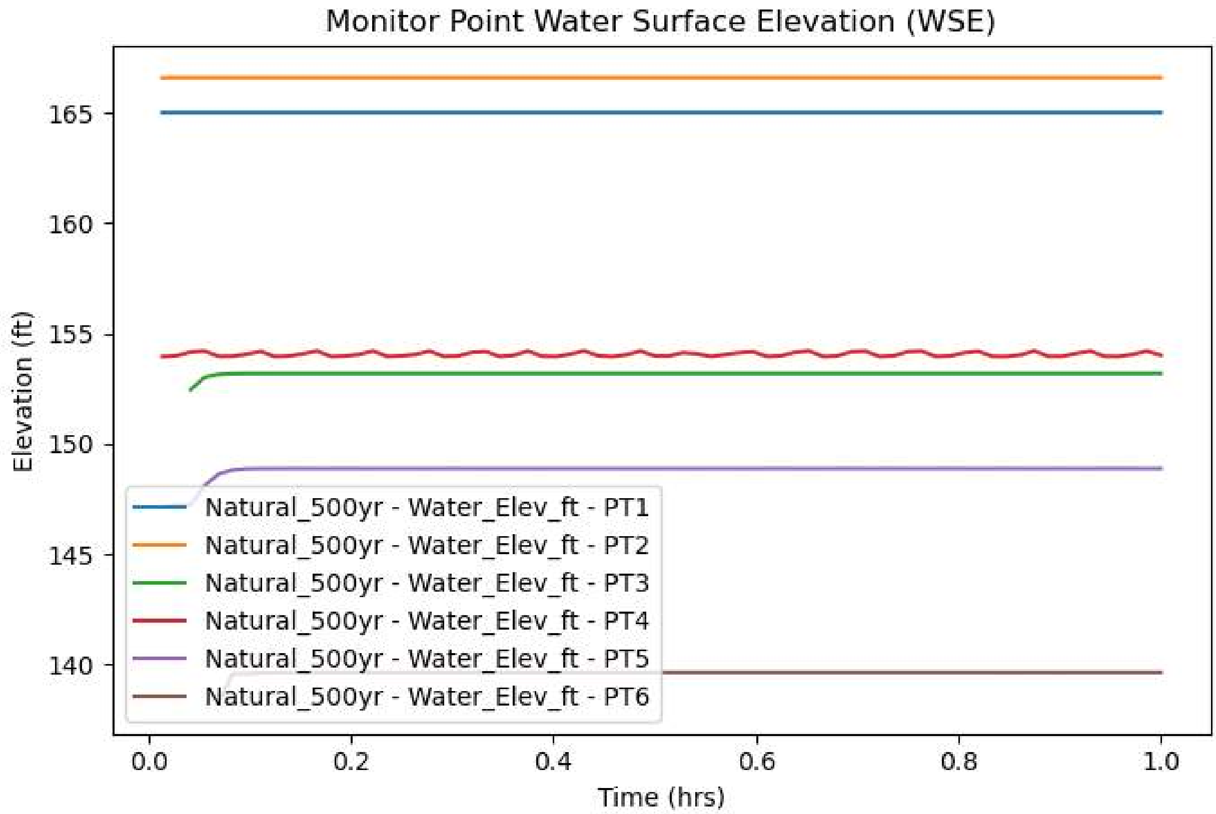


Figure 15. Natural 500-year Condition Monitor Line Plot

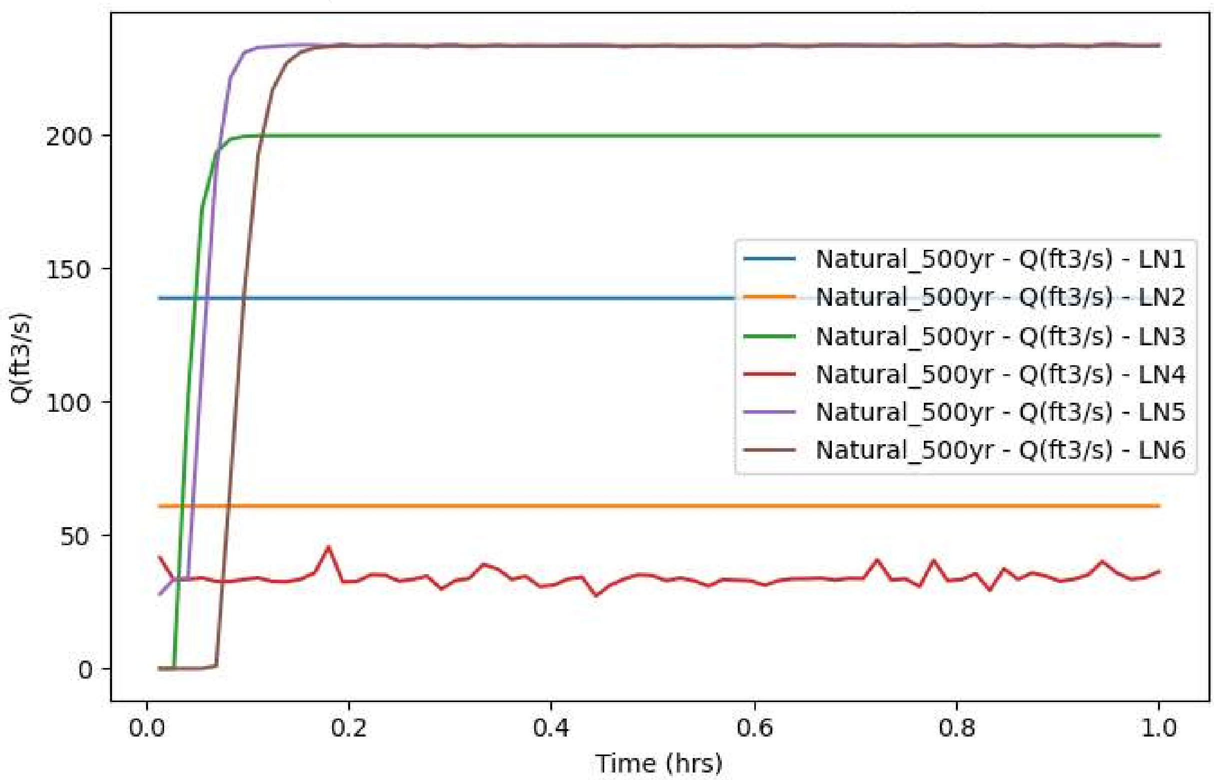


Figure 16. Natural 2080 Predicted 100-year Condition Monitor Point Plot

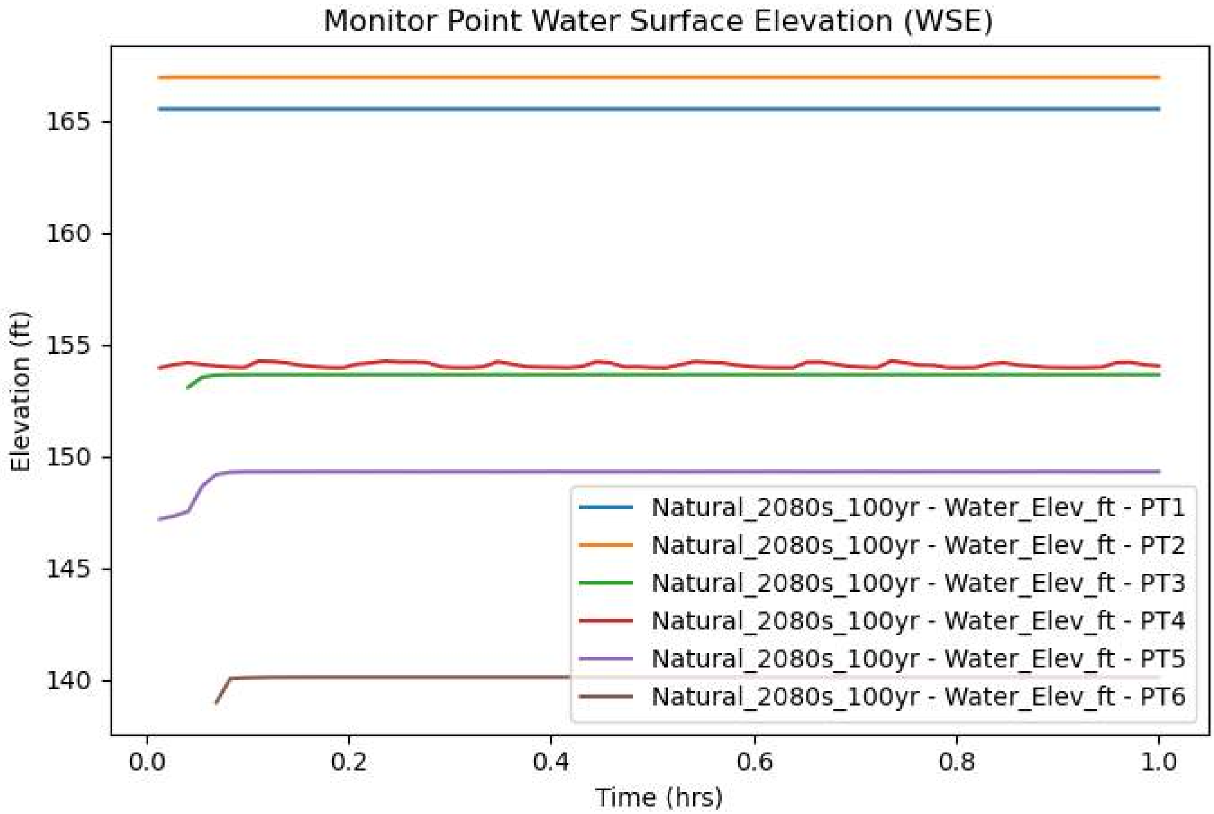


Figure 17. Natural 2080 Predicted 100-year Condition Monitor Line Plot

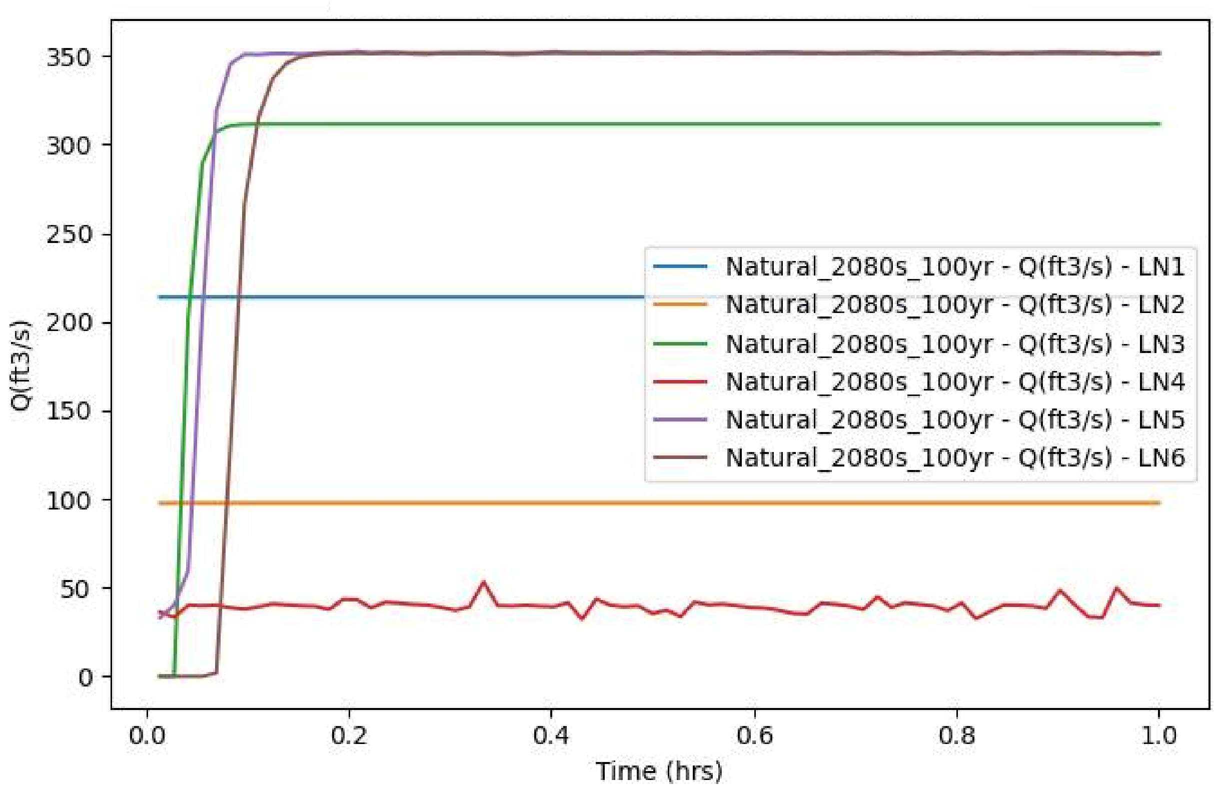


Figure 18. Proposed 2-year Condition Monitor Point Plot

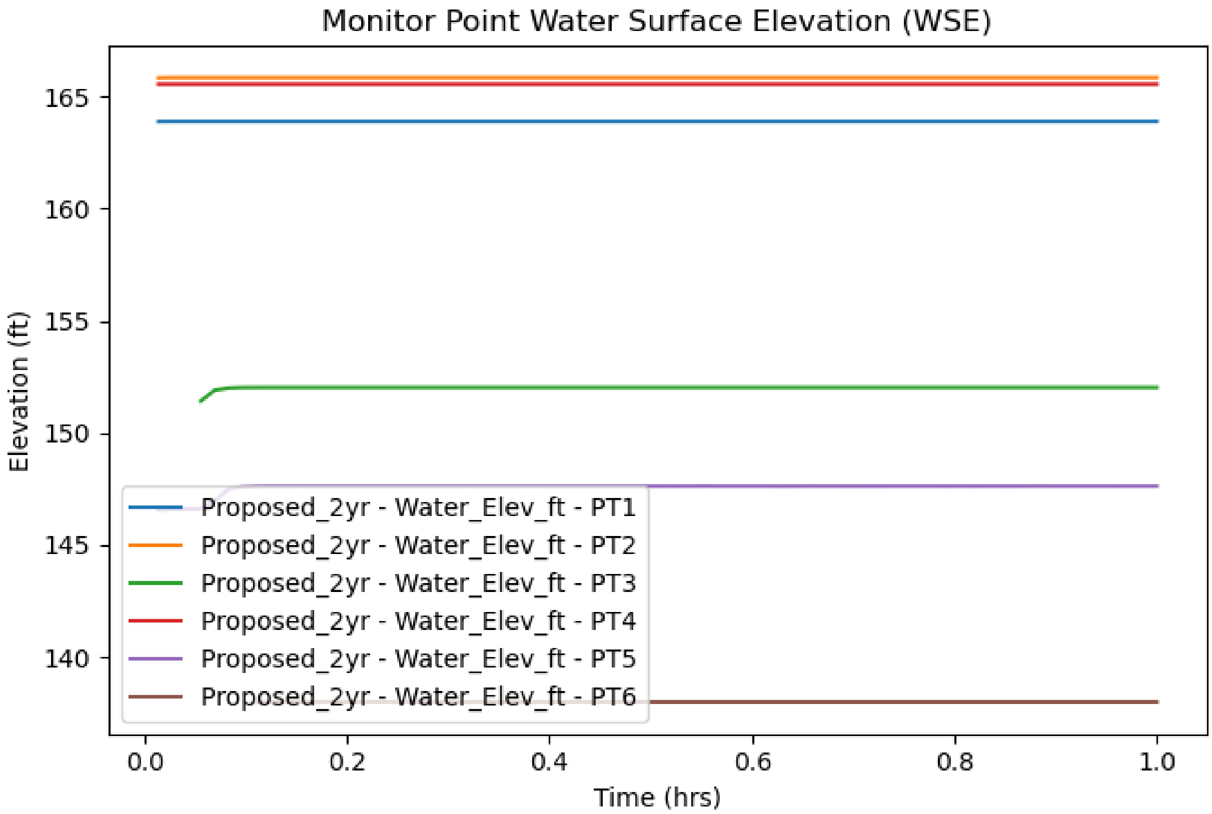


Figure 19. Proposed 2-year Condition Monitor Line Plot

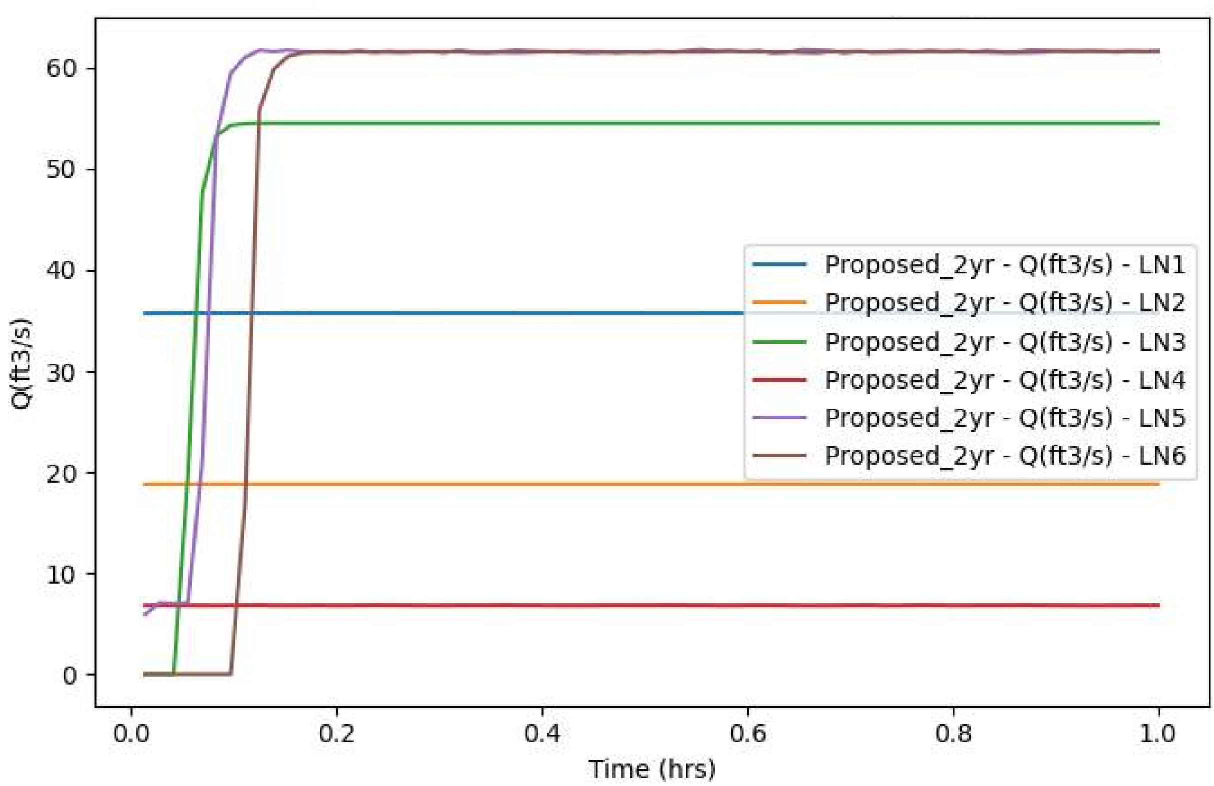


Figure 20. Proposed 100-year Condition Monitor Point Plot

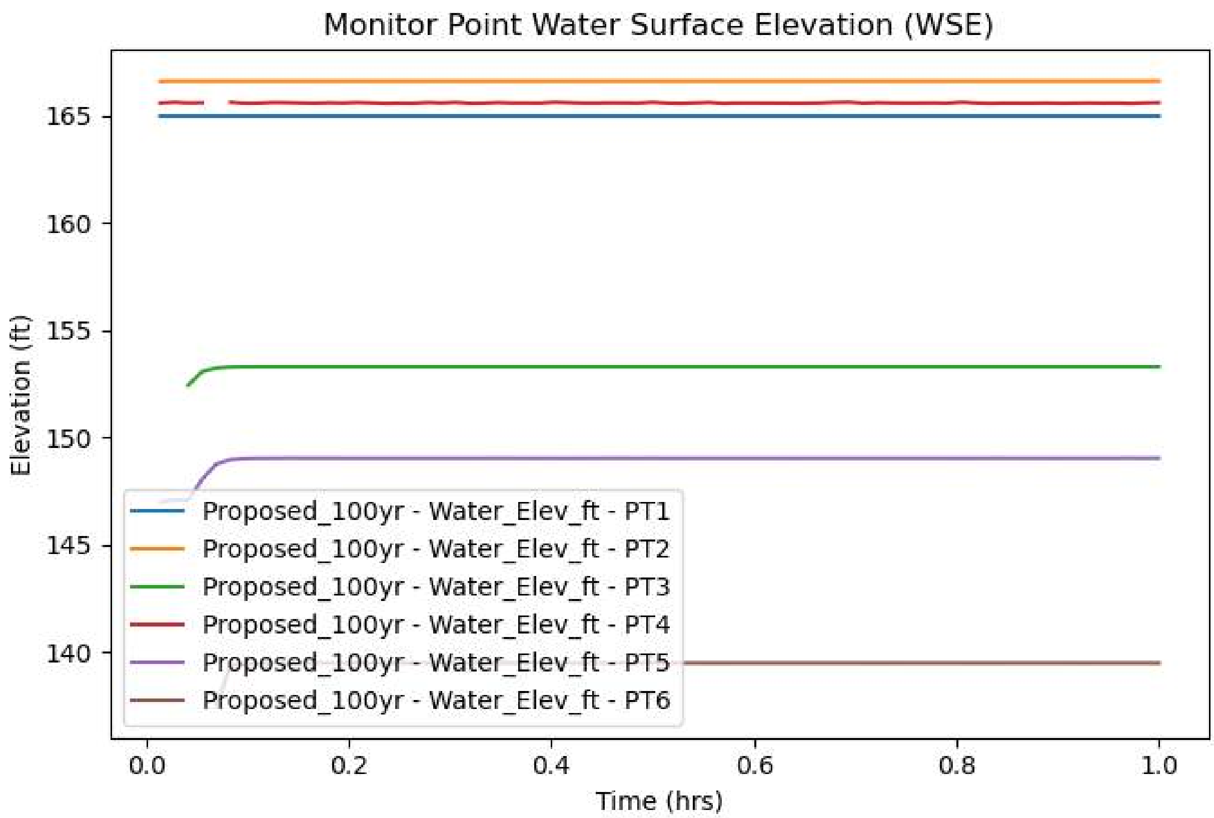


Figure 21. Proposed 100-year Condition Monitor Line Plot

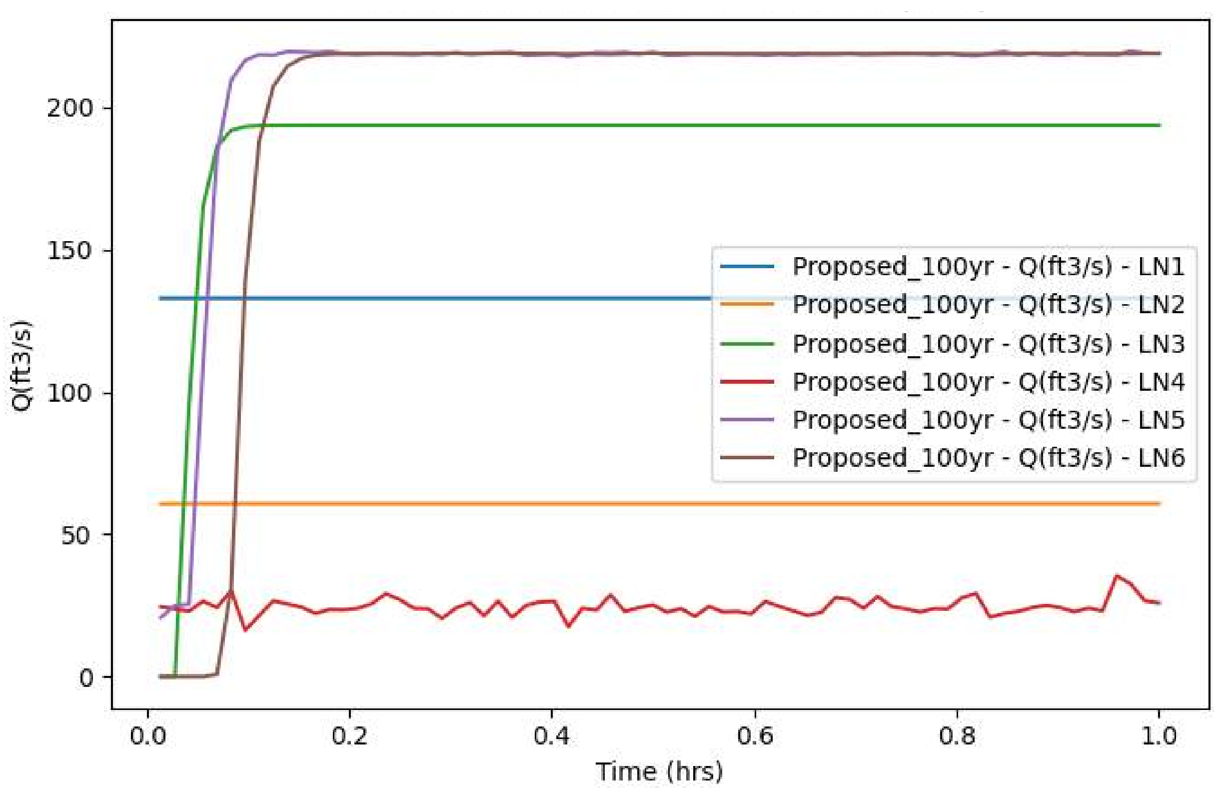


Figure 22. Proposed 500-year Condition Monitor Point Plot

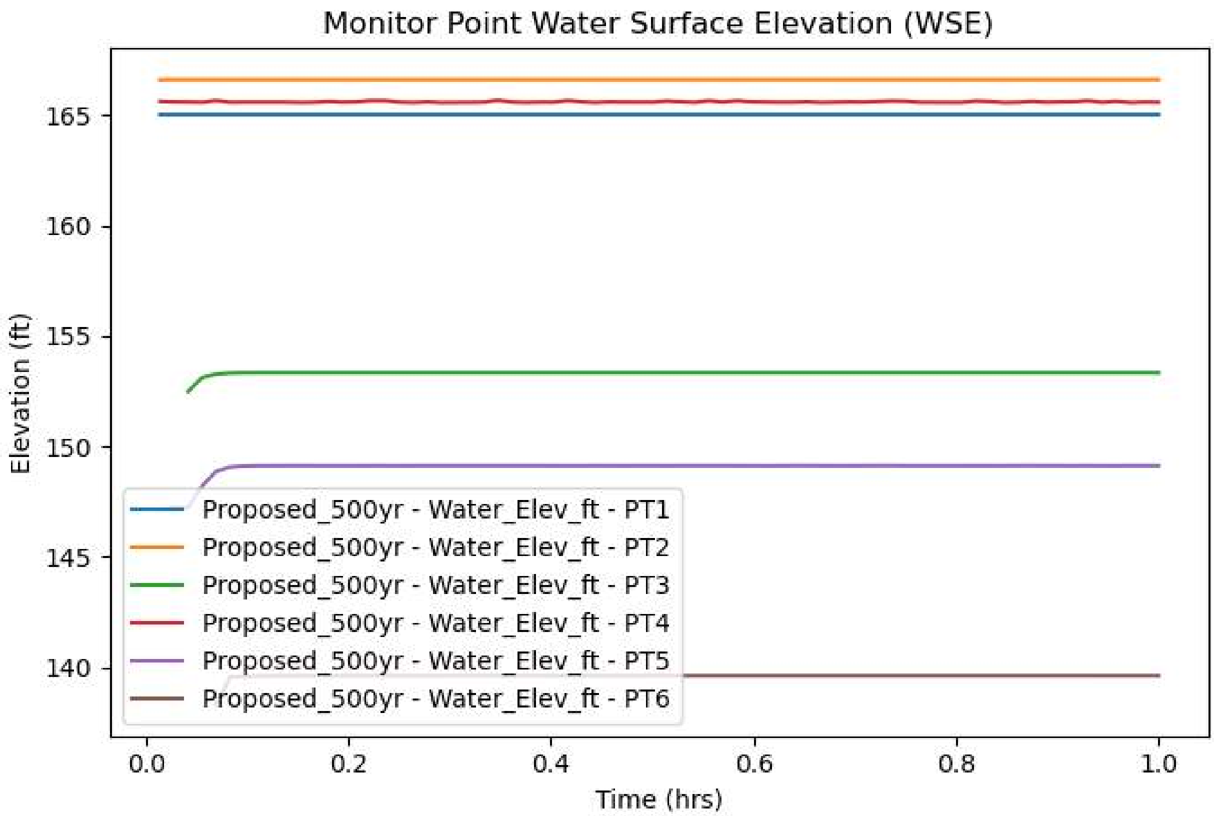


Figure 23. Proposed 500-year Condition Monitor Line Plot

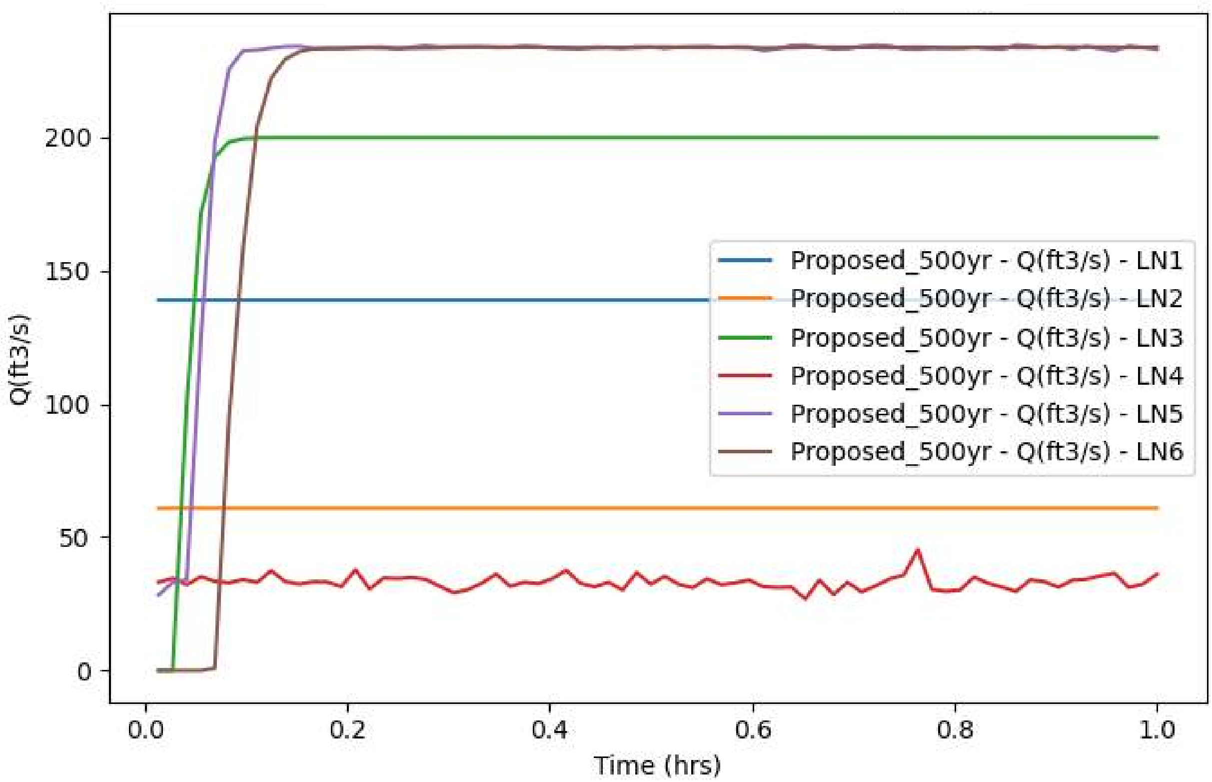


Figure 24. Proposed 2080 Predicted 100-year Condition Monitor Point Plot

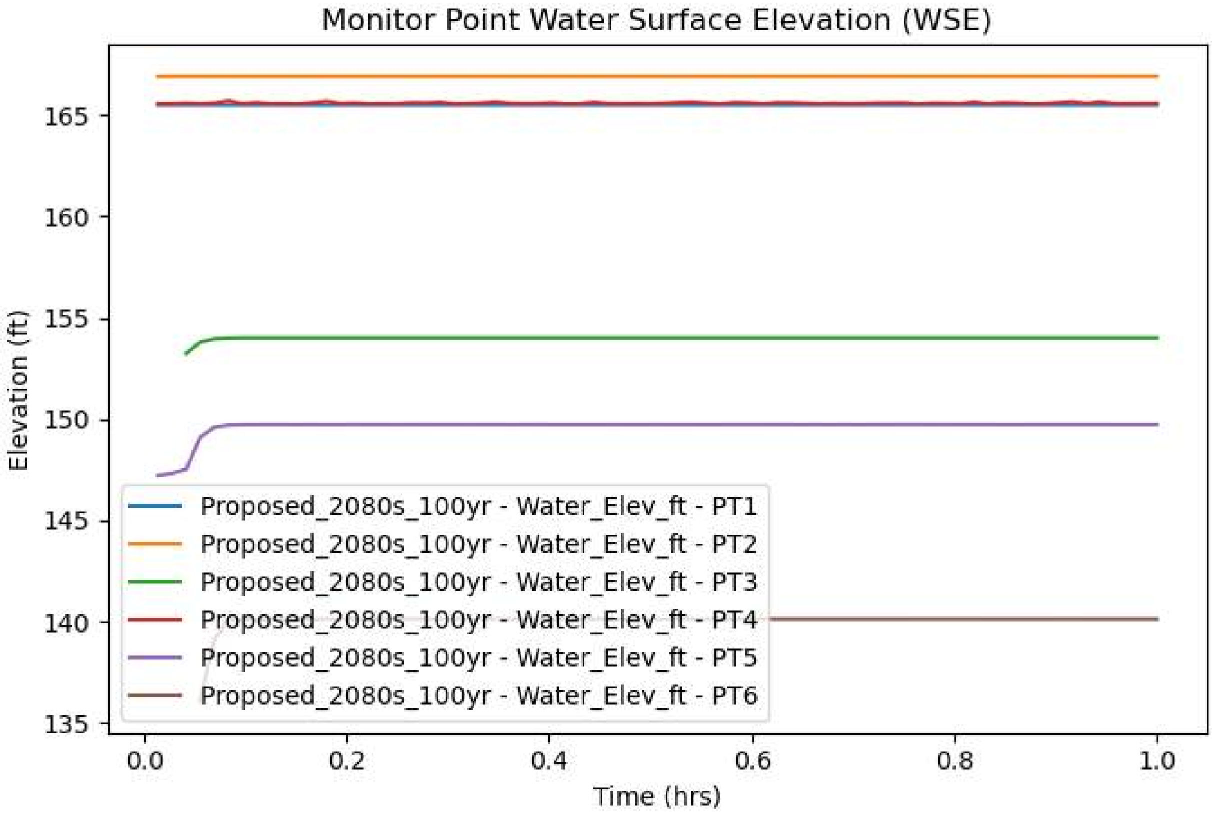
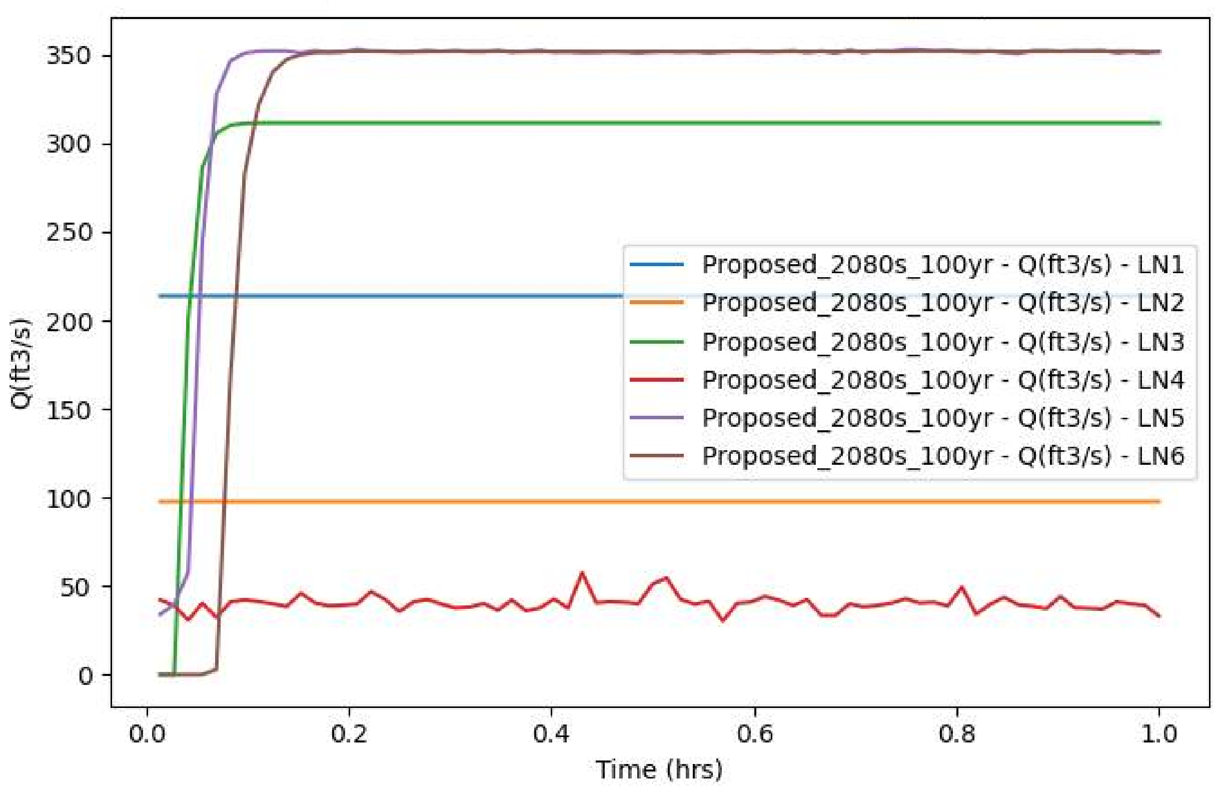


Figure 25. Proposed 2080 Predicted 100-year Condition Monitor Line Plot



Appendix K: Scour Calculations

Contraction Scour

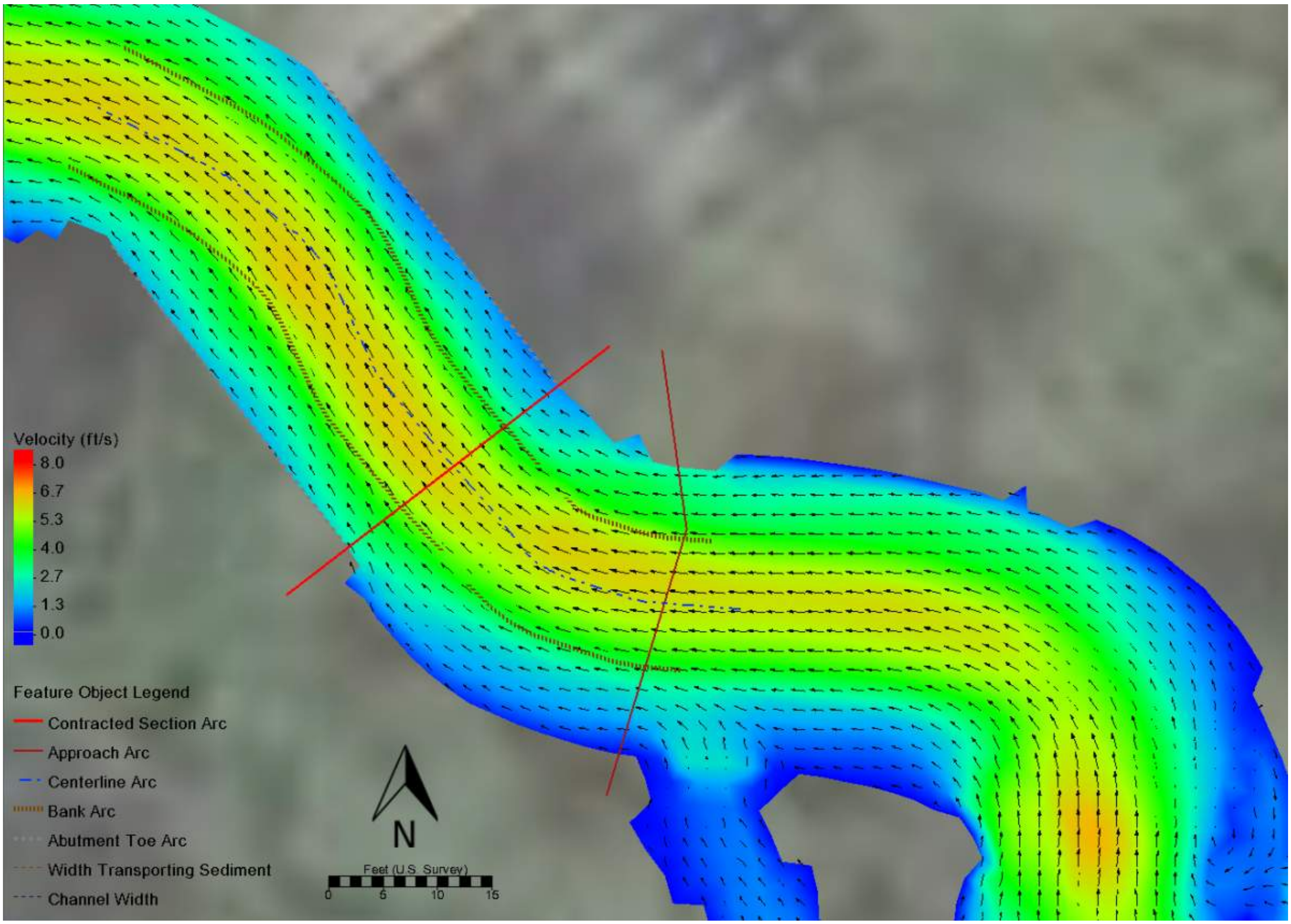


Figure 1. 100-year velocity results with bridge scour coverage arcs

Figure 2. 2-year critical velocity index map with bridge scour coverage arcs

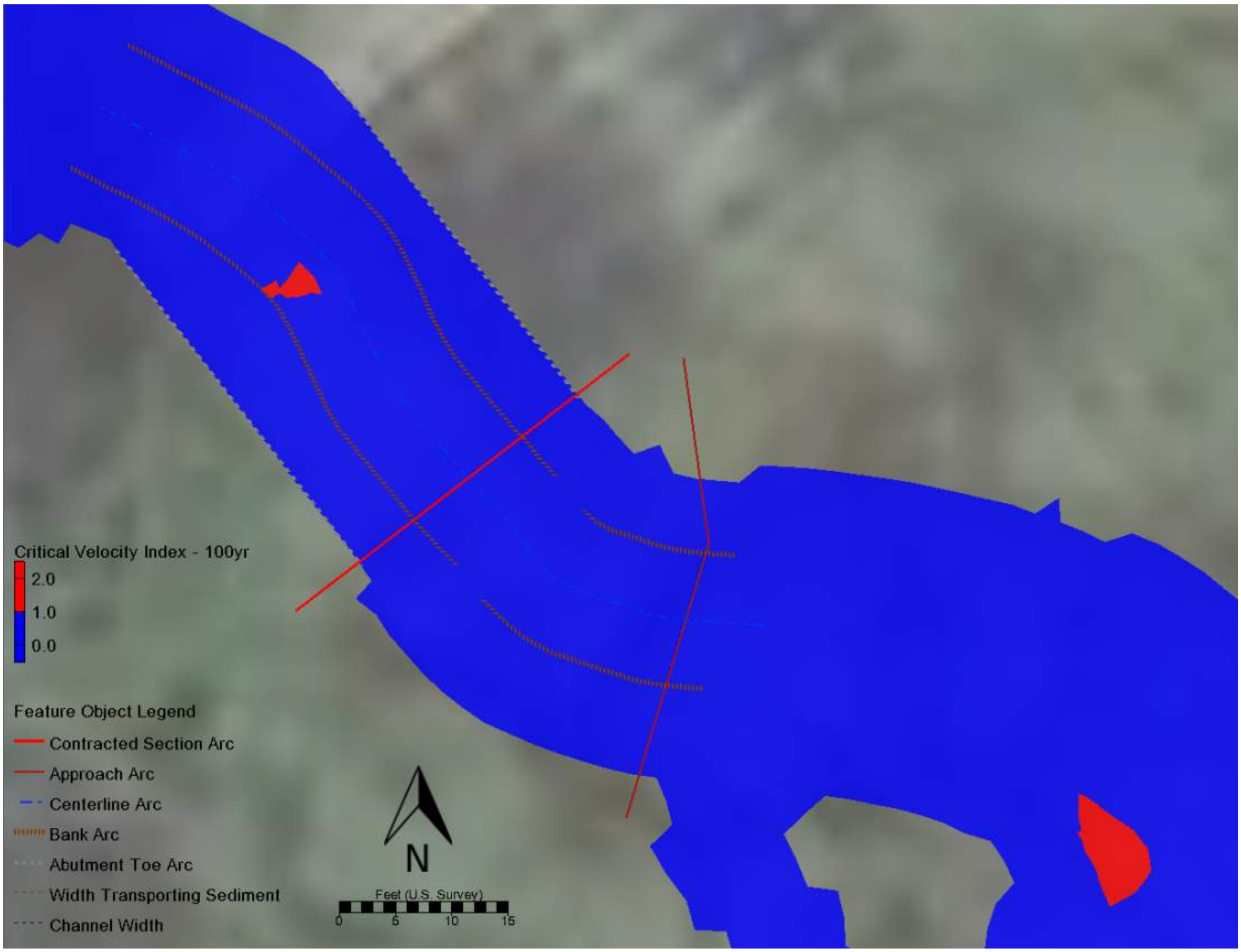


Figure 4. 100-year critical velocity index map with bridge scour coverage arcs

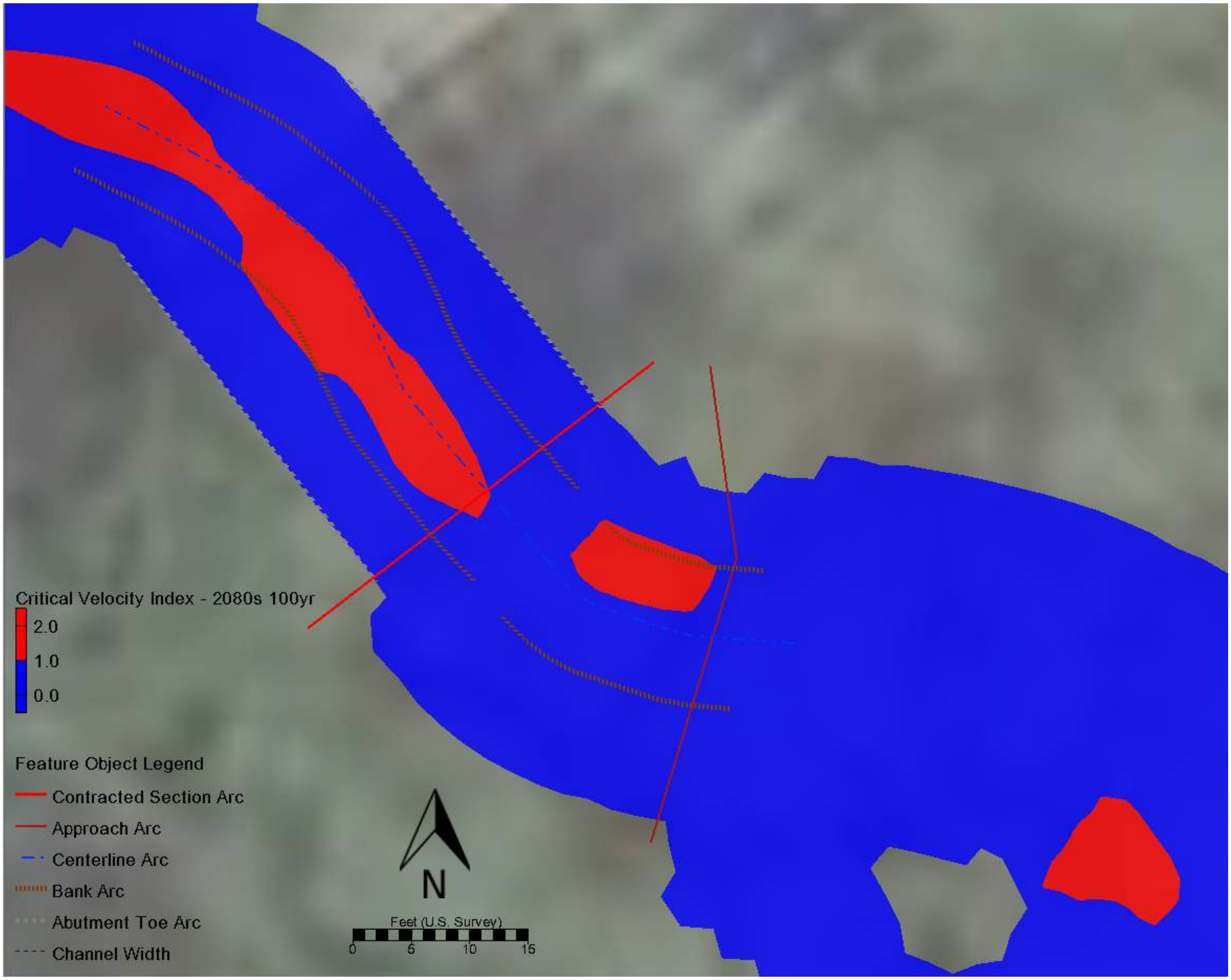


Figure 5. 2080s 100-year critical velocity index map with bridge scour coverage arcs

Computation Method: Clear-Water and Live-Bed Scour

Parameter	Value	Units	Notes
Input Parameters			
Average Depth Upstream of Contraction	1.12	ft	
D50	22.860000	mm	0.2 mm is the lower limit for ...
Average Velocity Upstream	3.95	ft/s	
Results of Scour Condition			
Critical velocity above which bed material of size D and s...	4.80	ft/s	
Contraction Scour Condition	Clear Water		
Clear Water Input Parameters			
Discharge in Contracted Section	54.20	cfs	
Bottom Width in Contracted Section	12.03	ft	Width should exclude pier wi...
Depth Prior to Scour in Contracted Section	1.10	ft	
Live Bed & Clear Water Input Parameters			
Temperature of Water	60.00	ºF	
Slope of Energy Grade Line at Approach Section	0.020777	ft/ft	
Discharge in Contracted Section	54.20	cfs	
Discharge Upstream that is Transporting Sediment	54.31	cfs	
Width in Contracted Section	12.03	ft	Remove widths occupied by ...
Width Upstream that is Transporting Sediment	12.27	ft	
Depth Prior to Scour in Contracted Section	1.10	ft	
Unit Weight of Water	62.40	lb/ft^3	
Unit Weight of Sediment	165.00	lb/ft^3	
Results of Clear Water Method			
Diameter of the smallest nontransportable particle in the b...	28.575000	mm	
Average Depth in Contracted Section after Scour	0.89	ft	
Scour Depth	-0.21	ft	Negative values imply 'zero'...
Results of Live Bed Method			
k1	0.640000		
Shear Velocity	0.87	ft/s	
Fall Velocity	1.64	ft/s	
Average Depth in Contracted Section after Scour	1.13	ft	
Scour Depth	0.03	ft	Negative values imply 'zero'...
Shear Applied to Bed by Live-Bed Scour	0.2267	lb/ft^2	
Shear Required for Movement of D50 Particle	0.3001	lb/ft^2	
Recommendations			
Recommended Scour Depth	-0.21	ft	Negative values imply 'zero'...

Figure 5. 2-year flow contraction scour results

Computation Method: Clear-Water and Live-Bed Scour

Parameter	Value	Units	Notes
Input Parameters			
Average Depth Upstream of Contraction	1.72	ft	
D50	22.860000	mm	0.2 mm is the lower limit for ...
Average Velocity Upstream	4.85	ft/s	
Results of Scour Condition			
Critical velocity above which bed material of size D and s...	5.15	ft/s	
Contraction Scour Condition	Clear Water		
Clear Water Input Parameters			
Discharge in Contracted Section	100.64	cfs	
Bottom Width in Contracted Section	12.03	ft	Width should exclude pier wi...
Depth Prior to Scour in Contracted Section	1.69	ft	
Live Bed & Clear Water Input Parameters			
Temperature of Water	60.00	ºF	
Slope of Energy Grade Line at Approach Section	0.020590	ft/ft	
Discharge in Contracted Section	100.64	cfs	
Discharge Upstream that is Transporting Sediment	102.13	cfs	
Width in Contracted Section	12.03	ft	Remove widths occupied by ...
Width Upstream that is Transporting Sediment	12.27	ft	
Depth Prior to Scour in Contracted Section	1.69	ft	
Unit Weight of Water	62.40	lb/ft^3	
Unit Weight of Sediment	165.00	lb/ft^3	
Results of Clear Water Method			
Diameter of the smallest nontransportable particle in the b...	28.575000	mm	
Average Depth in Contracted Section after Scour	1.51	ft	
Scour Depth	-0.18	ft	Negative values imply 'zero'...
Results of Live Bed Method			
k1	0.640000		
Shear Velocity	1.07	ft/s	
Fall Velocity	1.64	ft/s	
Average Depth in Contracted Section after Scour	1.72	ft	
Scour Depth	0.03	ft	Negative values imply 'zero'...
Shear Applied to Bed by Live-Bed Scour	0.3916	lb/ft^2	
Shear Required for Movement of D50 Particle	0.3001	lb/ft^2	
Recommendations			
Recommended Scour Depth	-0.18	ft	Negative values imply 'zero'...

Figure 6. 10-year flow contraction scour results

Computation Method: Clear-Water and Live-Bed Scour

Parameter	Value	Units	Notes
Input Parameters			
Average Depth Upstream of Contraction	1.98	ft	
D50	22.860000	mm	0.2 mm is the lower limit for ...
Average Velocity Upstream	5.11	ft/s	
Results of Scour Condition			
Critical velocity above which bed material of size D and s...	5.28	ft/s	
Contraction Scour Condition	Clear Water		
Clear Water Input Parameters			
Discharge in Contracted Section	122.59	cfs	
Bottom Width in Contracted Section	12.03	ft	Width should exclude pier wi...
Depth Prior to Scour in Contracted Section	1.95	ft	
Live Bed & Clear Water Input Parameters			
Temperature of Water	60.00	ºF	
Slope of Energy Grade Line at Approach Section	0.020424	ft/ft	
Discharge in Contracted Section	122.59	cfs	
Discharge Upstream that is Transporting Sediment	124.00	cfs	
Width in Contracted Section	12.03	ft	Remove widths occupied by ...
Width Upstream that is Transporting Sediment	12.27	ft	
Depth Prior to Scour in Contracted Section	1.95	ft	
Unit Weight of Water	62.40	lb/ft^3	
Unit Weight of Sediment	165.00	lb/ft^3	
Results of Clear Water Method			
Diameter of the smallest nontransportable particle in the b...	28.575000	mm	
Average Depth in Contracted Section after Scour	1.79	ft	
Scour Depth	-0.16	ft	Negative values imply 'zero' ...
Results of Live Bed Method			
k1	0.640000		
Shear Velocity	1.14	ft/s	
Fall Velocity	1.64	ft/s	
Average Depth in Contracted Section after Scour	1.98	ft	
Scour Depth	0.03	ft	Negative values imply 'zero' ...
Shear Applied to Bed by Live-Bed Scour	0.4570	lb/ft^2	
Shear Required for Movement of D50 Particle	0.3001	lb/ft^2	
Recommendations			
Recommended Scour Depth	-0.16	ft	Negative values imply 'zero' ...

Figure 7. 25-year flow contraction scour results

Computation Method: Clear-Water and Live-Bed Scour

Parameter	Value	Units	Notes
Input Parameters			
Average Depth Upstream of Contraction	2.25	ft	
D50	22.860000	mm	0.2 mm is the lower limit for ...
Average Velocity Upstream	5.30	ft/s	
Results of Scour Condition			
Critical velocity above which bed material of size D and s...	5.39	ft/s	
Contraction Scour Condition	Clear Water		
Clear Water Input Parameters			
Discharge in Contracted Section	146.19	cfs	
Bottom Width in Contracted Section	12.03	ft	Width should exclude pier wi...
Depth Prior to Scour in Contracted Section	2.22	ft	
Live Bed & Clear Water Input Parameters			
Temperature of Water	60.00	ºF	
Slope of Energy Grade Line at Approach Section	0.020391	ft/ft	
Discharge in Contracted Section	146.19	cfs	
Discharge Upstream that is Transporting Sediment	146.57	cfs	
Width in Contracted Section	12.03	ft	Remove widths occupied by ...
Width Upstream that is Transporting Sediment	12.27	ft	
Depth Prior to Scour in Contracted Section	2.22	ft	
Unit Weight of Water	62.40	lb/ft^3	
Unit Weight of Sediment	165.00	lb/ft^3	
Results of Clear Water Method			
Diameter of the smallest nontransportable particle in the b...	28.575000	mm	
Average Depth in Contracted Section after Scour	2.08	ft	
Scour Depth	-0.14	ft	Negative values imply 'zero' ...
Results of Live Bed Method			
k1	0.640000		
Shear Velocity	1.22	ft/s	
Fall Velocity	1.64	ft/s	
Average Depth in Contracted Section after Scour	2.28	ft	
Scour Depth	0.06	ft	Negative values imply 'zero' ...
Shear Applied to Bed by Live-Bed Scour	0.5159	lb/ft^2	
Shear Required for Movement of D50 Particle	0.3001	lb/ft^2	
Recommendations			
Recommended Scour Depth	-0.14	ft	Negative values imply 'zero' ...

Figure 8. 50-year flow contraction scour results

Computation Method: Clear-Water and Live-Bed Scour

Parameter	Value	Units	Notes
Input Parameters			
Average Depth Upstream of Contraction	2.32	ft	
D50	22.860000	mm	0.2 mm is the lower limit for …
Average Velocity Upstream	5.34	ft/s	
Results of Scour Condition			
Critical velocity above which bed material of size D and s…	5.42	ft/s	
Contraction Scour Condition	Clear Water		
Clear Water Input Parameters			
Discharge in Contracted Section	152.09	cfs	
Bottom Width in Contracted Section	12.03	ft	Width should exclude pier wi…
Depth Prior to Scour in Contracted Section	2.28	ft	
Live Bed & Clear Water Input Parameters			
Temperature of Water	60.00	ºF	
Slope of Energy Grade Line at Approach Section	0.020408	ft/ft	
Discharge in Contracted Section	152.09	cfs	
Discharge Upstream that is Transporting Sediment	152.13	cfs	
Width in Contracted Section	12.03	ft	Remove widths occupied by …
Width Upstream that is Transporting Sediment	12.27	ft	
Depth Prior to Scour in Contracted Section	2.28	ft	
Unit Weight of Water	62.40	lb/ft^3	
Unit Weight of Sediment	165.00	lb/ft^3	
Results of Clear Water Method			
Diameter of the smallest nontransportable particle in the b…	28.575000	mm	
Average Depth in Contracted Section after Scour	2.15	ft	
Scour Depth	-0.13	ft	Negative values imply 'zero' …
Results of Live Bed Method			
k1	0.640000		
Shear Velocity	1.24	ft/s	
Fall Velocity	1.64	ft/s	
Average Depth in Contracted Section after Scour	2.35	ft	
Scour Depth	0.07	ft	Negative values imply 'zero' …
Shear Applied to Bed by Live-Bed Scour	0.5296	lb/ft^2	
Shear Required for Movement of D50 Particle	0.3001	lb/ft^2	
Recommendations			
Recommended Scour Depth	-0.13	ft	Negative values imply 'zero' …

Figure 9. 100-year flow contraction scour results

Computation Method: Clear-Water and Live-Bed Scour

Parameter	Value	Units	Notes
Input Parameters			
Average Depth Upstream of Contraction	2.37	ft	
D50	22.860000	mm	0.2 mm is the lower limit for ...
Average Velocity Upstream	5.36	ft/s	
Results of Scour Condition			
Critical velocity above which bed material of size D and s...	5.44	ft/s	
Contraction Scour Condition	Clear Water		
Clear Water Input Parameters			
Discharge in Contracted Section	155.94	cfs	
Bottom Width in Contracted Section	12.03	ft	Width should exclude pier wi...
Depth Prior to Scour in Contracted Section	2.33	ft	
Live Bed & Clear Water Input Parameters			
Temperature of Water	60.00	ºF	
Slope of Energy Grade Line at Approach Section	0.020410	ft/ft	
Discharge in Contracted Section	155.94	cfs	
Discharge Upstream that is Transporting Sediment	155.73	cfs	
Width in Contracted Section	12.03	ft	Remove widths occupied by ...
Width Upstream that is Transporting Sediment	12.27	ft	
Depth Prior to Scour in Contracted Section	2.33	ft	
Unit Weight of Water	62.40	lb/ft^3	
Unit Weight of Sediment	165.00	lb/ft^3	
Results of Clear Water Method			
Diameter of the smallest nontransportable particle in the b...	28.575000	mm	
Average Depth in Contracted Section after Scour	2.20	ft	
Scour Depth	-0.13	ft	Negative values imply 'zero' ...
Results of Live Bed Method			
k1	0.640000		
Shear Velocity	1.25	ft/s	
Fall Velocity	1.64	ft/s	
Average Depth in Contracted Section after Scour	2.40	ft	
Scour Depth	0.07	ft	Negative values imply 'zero' ...
Shear Applied to Bed by Live-Bed Scour	0.5380	lb/ft^2	
Shear Required for Movement of D50 Particle	0.3001	lb/ft^2	
Recommendations			
Recommended Scour Depth	-0.13	ft	Negative values imply 'zero' ...

Figure 10. 500-year flow contraction scour results

Computation Method: Clear-Water and Live-Bed Scour

Parameter	Value	Units	Notes
Input Parameters			
Average Depth Upstream of Contraction	3.11	ft	
D50	22.860000	mm	0.2 mm is the lower limit for ...
Average Velocity Upstream	5.65	ft/s	
Results of Scour Condition			
Critical velocity above which bed material of size D and s...	5.69	ft/s	
Contraction Scour Condition	Clear Water		
Clear Water Input Parameters			
Discharge in Contracted Section	222.99	cfs	
Bottom Width in Contracted Section	12.02	ft	Width should exclude pier wi...
Depth Prior to Scour in Contracted Section	3.01	ft	
Live Bed & Clear Water Input Parameters			
Temperature of Water	60.00	ºF	
Slope of Energy Grade Line at Approach Section	0.021159	ft/ft	
Discharge in Contracted Section	222.99	cfs	
Discharge Upstream that is Transporting Sediment	215.19	cfs	
Width in Contracted Section	12.02	ft	Remove widths occupied by ...
Width Upstream that is Transporting Sediment	12.27	ft	
Depth Prior to Scour in Contracted Section	3.01	ft	
Unit Weight of Water	62.40	lb/ft^3	
Unit Weight of Sediment	165.00	lb/ft^3	
Results of Clear Water Method			
Diameter of the smallest nontransportable particle in the b...	28.575000	mm	
Average Depth in Contracted Section after Scour	2.99	ft	
Scour Depth	-0.02	ft	Negative values imply 'zero'...
Results of Live Bed Method			
k1	0.640000		
Shear Velocity	1.45	ft/s	
Fall Velocity	1.64	ft/s	
Average Depth in Contracted Section after Scour	3.24	ft	
Scour Depth	0.24	ft	Negative values imply 'zero'...
Shear Applied to Bed by Live-Bed Scour	0.6668	lb/ft^2	
Shear Required for Movement of D50 Particle	0.3001	lb/ft^2	
Recommendations			
Recommended Scour Depth	-0.02	ft	Negative values imply 'zero'...

Figure 11. 2080 Projected 100-year flow contraction scour results

Abutment Scour

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Channel)		
Abutment Type	Vertical-wall abutment		
Unit Discharge, Upstream in Main Channel (q1)	4.43	cfs/ft	
Unit Discharge in Constricted Area (q2)	4.50	cfs/ft	
D50	22.860000	mm	0.2 mm is the lower limit for coh...
Upstream Flow Depth	1.12	ft	
Define Shear Stress of Floodplain	☐		
Flow Depth prior to Scour	1.53	ft	Depth at Abutment Toe
Results			
q2 / q1	1.02		
Average Velocity Upstream	3.95	ft/s	
Critical Velocity above which Bed Materal of Size D and Sm...	4.80	ft/s	
Scour Condition	Clear Water		
Scour Condition	a (Main Channel)		
Amplification Factor	1.29		
Flow Depth including Contraction Scour	0.96	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour	1.24	ft	Including the long-term scour de...
Scour Hole Depth	-0.29	ft	Negative values imply 'zero' sco...
Scour Hole			
Angle of Repose	44.00	degrees	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will ...

Figure 12. 2-year flow Abutment Scour Results

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Channel)		
Abutment Type	Vertical-wall abutment		
Unit Discharge, Upstream in Main Channel (q1)	8.33	cfs/ft	
Unit Discharge in Constricted Area (q2)	8.37	cfs/ft	
D50	22.860000	mm	0.2 mm is the lower limit for coh...
Upstream Flow Depth	1.72	ft	
Define Shear Stress of Floodplain	☐		
Flow Depth prior to Scour	2.13	ft	Depth at Abutment Toe
Results			
q2 / q1	1.00		
Average Velocity Upstream	4.85	ft/s	
Critical Velocity above which Bed Materal of Size D and Sm...	5.15	ft/s	
Scour Condition	Clear Water		
Scour Condition	a (Main Channel)		
Amplification Factor	1.22		
Flow Depth including Contraction Scour	1.64	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour	2.00	ft	Including the long-term scour de...
Scour Hole Depth	-0.13	ft	Negative values imply 'zero' sco...
Scour Hole			
Angle of Repose	44.00	degrees	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will ...

Figure 13. 10-year flow Abutment Scour Results

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Channel)		
Abutment Type	Vertical-wall abutment		
Unit Discharge, Upstream in Main Channel (q1)	10.11	cfs/ft	
Unit Discharge in Constricted Area (q2)	10.19	cfs/ft	
D50	22.860000	mm	0.2 mm is the lower limit for coh…
Upstream Flow Depth	1.98	ft	
Define Shear Stress of Floodplain	☐		
Flow Depth prior to Scour	2.39	ft	Depth at Abutment Toe
Results			
q2 / q1	1.01		
Average Velocity Upstream	5.11	ft/s	
Critical Velocity above which Bed Materal of Size D and Sm…	5.28	ft/s	
Scour Condition	Clear Water		
Scour Condition	a (Main Channel)		
Amplification Factor	1.24		
Flow Depth including Contraction Scour	1.94	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour	2.40	ft	Including the long-term scour de…
Scour Hole Depth	0.01	ft	Negative values imply 'zero' sco…
Scour Hole			
Angle of Repose	44.00	degrees	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will …
Scour Hole Bottom Width	0.00	ft	
Scour Hole Top Width	0.01	ft	

Figure 14. 25-year flow Abutment Scour Results

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Channel)		
Abutment Type	Vertical-wall abutment		
Unit Discharge, Upstream in Main Channel (q1)	11.95	cfs/ft	
Unit Discharge in Constricted Area (q2)	12.15	cfs/ft	
D50	22.860000	mm	0.2 mm is the lower limit for coh…
Upstream Flow Depth	2.25	ft	
Define Shear Stress of Floodplain	☐		
Flow Depth prior to Scour	2.65	ft	Depth at Abutment Toe
Results			
q2 / q1	1.02		
Average Velocity Upstream	5.30	ft/s	
Critical Velocity above which Bed Materal of Size D and Sm…	5.39	ft/s	
Scour Condition	Clear Water		
Scour Condition	a (Main Channel)		
Amplification Factor	1.29		
Flow Depth including Contraction Scour	2.25	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour	2.91	ft	Including the long-term scour de…
Scour Hole Depth	0.26	ft	Negative values imply 'zero' sco…
Scour Hole			
Angle of Repose	44.00	degrees	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will …
Scour Hole Bottom Width	0.00	ft	
Scour Hole Top Width	0.27	ft	

Figure 15. 50-year flow Abutment Scour Results

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Channel)		
Abutment Type	Vertical-wall abutment		
Unit Discharge, Upstream in Main Channel (q1)	12.40	cfs/ft	
Unit Discharge in Constricted Area (q2)	12.64	cfs/ft	
D50	22.860000	mm	0.2 mm is the lower limit for coh...
Upstream Flow Depth	2.32	ft	
Define Shear Stress of Floodplain	☐		
Flow Depth prior to Scour	2.72	ft	Depth at Abutment Toe
Results			
q2 / q1	1.02		
Average Velocity Upstream	5.34	ft/s	
Critical Velocity above which Bed Materal of Size D and Sm...	5.42	ft/s	
Scour Condition	Clear Water		
Scour Condition	a (Main Channel)		
Amplification Factor	1.30		
Flow Depth including Contraction Scour	2.33	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour	3.04	ft	Including the long-term scour de...
Scour Hole Depth	0.32	ft	Negative values imply 'zero' sco...
Scour Hole			
Angle of Repose	44.00	degrees	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will ...
Scour Hole Bottom Width	0.00	ft	
Scour Hole Top Width	0.33	ft	

Figure 16. 100-year flow Abutment Scour Results

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Channel)		
Abutment Type	Vertical-wall abutment		
Unit Discharge, Upstream in Main Channel (q1)	12.70	cfs/ft	
Unit Discharge in Constricted Area (q2)	12.96	cfs/ft	
D50	22.860000	mm	0.2 mm is the lower limit for coh...
Upstream Flow Depth	2.37	ft	
Define Shear Stress of Floodplain	☐		
Flow Depth prior to Scour	2.76	ft	Depth at Abutment Toe
Results			
q2 / q1	1.02		
Average Velocity Upstream	5.36	ft/s	
Critical Velocity above which Bed Materal of Size D and Sm...	5.44	ft/s	
Scour Condition	Clear Water		
Scour Condition	a (Main Channel)		
Amplification Factor	1.31		
Flow Depth including Contraction Scour	2.38	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour	3.13	ft	Including the long-term scour de...
Scour Hole Depth	0.37	ft	Negative values imply 'zero' sco...
Scour Hole			
Angle of Repose	44.00	degrees	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will ...
Scour Hole Bottom Width	0.00	ft	
Scour Hole Top Width	0.38	ft	

Figure 17. 500-year flow Abutment Scour Results

Computation Method: NCHRP

Parameter	Value	Units	Notes
Input Parameters			
Scour Condition	Compute		
Scour Condition Location	Type a (Main Channel)		
Abutment Type	Vertical-wall abutment		
Unit Discharge, Upstream in Main Channel (q1)	17.55	cfs/ft	
Unit Discharge in Constricted Area (q2)	18.54	cfs/ft	
D50	22.860000	mm	0.2 mm is the lower limit for coh...
Upstream Flow Depth	3.11	ft	
Define Shear Stress of Floodplain			
Flow Depth prior to Scour	3.45	ft	Depth at Abutment Toe
Results			
q2 / q1	1.06		
Average Velocity Upstream	5.65	ft/s	
Critical Velocity above which Bed Materal of Size D and Sm...	5.69	ft/s	
Scour Condition	Clear Water		
Scour Condition	a (Main Channel)		
Amplification Factor	1.50		
Flow Depth including Contraction Scour	3.24	ft	
Scour depth from Long-Term Degradation calculations	0.00	ft	
Maximum Flow Depth including Abutment Scour	4.84	ft	Including the long-term scour de...
Scour Hole Depth	1.39	ft	Negative values imply 'zero' sco...
Scour Hole			
Angle of Repose	44.00	degrees	
Ratio of Bottom Width of Scour Hole to Scour Hole Depth	0.00		1.0 means the bottom width will ...
Scour Hole Bottom Width	0.00	ft	
Scour Hole Top Width	1.44	ft	

Figure 18. 2080 Projected 100-year flow Left Abutment Scour Results

Bend Scour

Calculated using the Maynords equation found in the Bureau of Reclamation spreadsheet

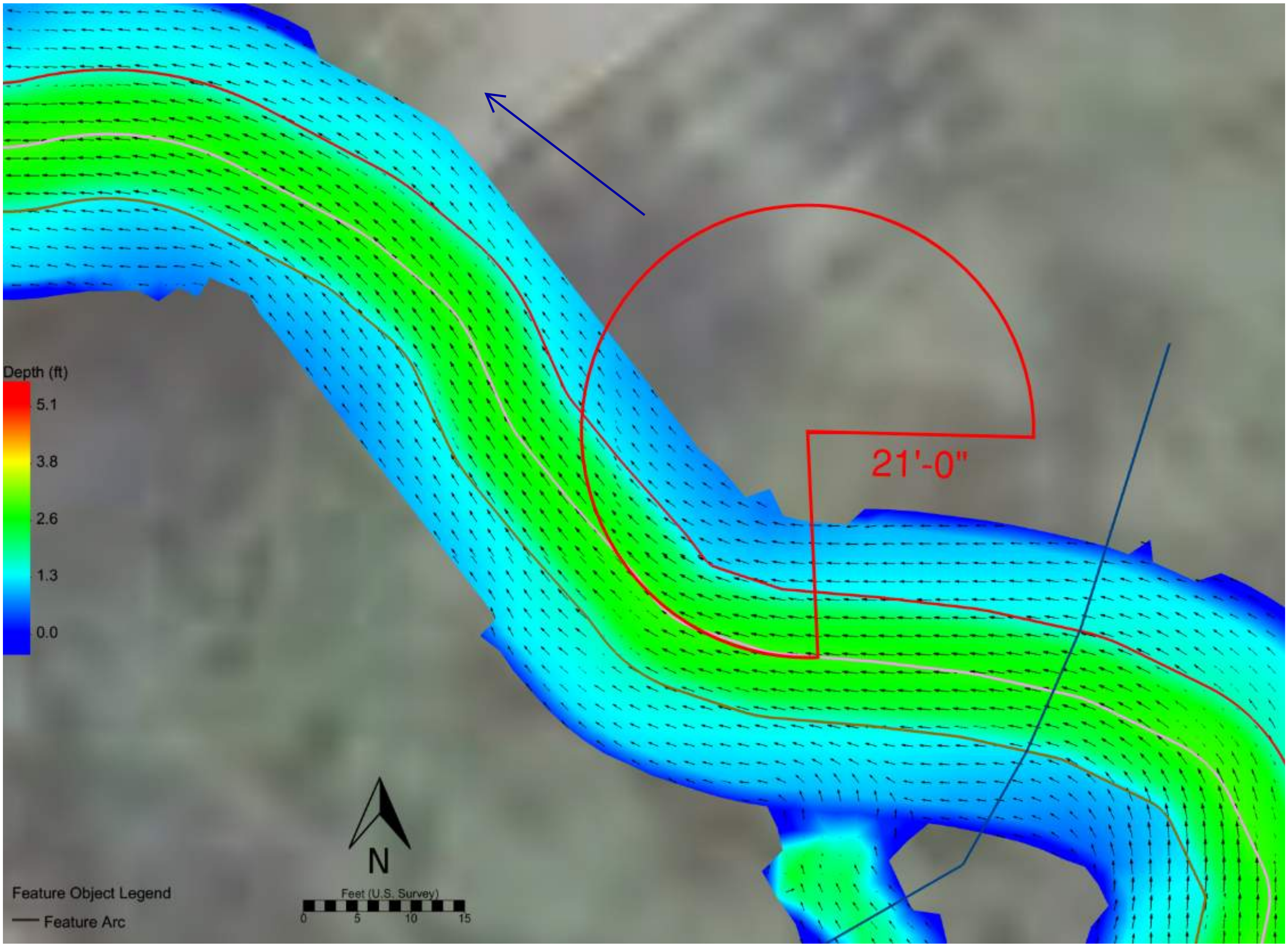


Figure 19. 100-year velocity results with location where bend scour values were taken

Channel Scour Analysis - Bend Scour

Project:	Y-12554	Site:	991572
Watercourse:	NE Dogfish	Analyst:	0
Checked By:	0	Latest Revision:	0

Description:

- This workbook contains calculations to estimate the amount of scour associated with the bend in a channel.
- The Thorne Equation is taken from Reference 3, the Maynord equation is taken from Reference 3, the NEH equation is taken from Reference 4, the Zeller equation is taken from Reference

Bend Scour - Thorne Equation

Input

	Hydraulic Scenario			
	100 year flow	2080 prj 100 yr flow	0	0
Y_{us} (ft)	2.3	3.1	0.0	0.0
TW_{us} (ft)	12.0	12.0	0.0	0.0
RC (ft)	21.0	21.0	0.0	0.0

Calculations

RC/TW_{us}	1.8	1.8	#DIV/0!	#DIV/0!
Check Range	2.1	2.1	#DIV/0!	#DIV/0!
z_b (ft)	2.9	3.9	#DIV/0!	#DIV/0!

Bend Scour - Maynord Equation

Input

	Hydraulic Scenario			
	100 year flow	2080 prj 100 yr flow	0	0
Y_{us} (ft)	2.3	3.1	0	0
TW_{us} (ft)	12	12	0	0
RC (ft)	21	21	0	0

Calculations

RC/TW_{us}	1.8	1.8	#DIV/0!	#DIV/0!
Check Range	1.8	1.8	#DIV/0!	#DIV/0!
TW_{us}/y_{us}	5.2	3.9	#DIV/0!	#DIV/0!
Check Range	20.0	20.0	#DIV/0!	#DIV/0!
z_b (ft)	2.0	2.7	#DIV/0!	#DIV/0!

Bend Scour - National Engineering Handbook

Input

	Hydraulic Scenario			
	100 year flow	2080 prj 100 yr flow	0	0
Y_{us} (ft)	2.3	3.1	0	0
TW_{us} (ft)	12	12	0	0
RC (ft)	21	21	0	0

Calculations

TW_{us}/RC	0.6	0.6	#DIV/0!	#DIV/0!
z_b (ft)	6.9	9.3	#DIV/0!	#DIV/0!

Channel Scour Analysis - Bend Scour

Project:	Y-12554	Site:	991572
Watercourse:	NE Dogfish	Analyst:	0
Checked By:	0	Latest Revision:	0

Bend Scour - Zeller Equation

Input

	Hydraulic Scenario			
	100 year flow	2080 prj 100 yr flow	0	0
TW_{us} (ft)	12	12	0	0
RC (ft)	21	21	0	0
$Y_{h\ us}$ (ft)	2.3	3.1	0	0
$Y_{max\ us}$ (ft)	0.0	0.0	0.0	0.0
S_o (ft/ft)	0.00000	0.00000	0.00000	0.00000
V_{us} (ft/s)	0.0	0.0	0.0	0.0

Calculations

TW_{us}/RC	0.57	0.57	#DIV/0!	#DIV/0!
z_b (ft)	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

Summary of Bend Scour

	Hydraulic Scenario			
	100 year flow	2080 prj 100 yr flow	0	0
Thorne	2.9	3.9	#DIV/0!	#DIV/0!
Maynord	2.0	2.7	#DIV/0!	#DIV/0!
Zeller	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
NEH-Max	6.9	9.3	#DIV/0!	#DIV/0!

Average Bend Scour

Bend Scour	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!